TECHNICAL MEMORANDUM 1

Regulatory Requirements

(Text to be prepared by Stafford County DOU)

TECHNICAL MEMORANDUM 2

Water Demands

Prepared for: Stafford County Department of Utilities
Prepare by: O'Brien & Gere
Date: November 2004

This technical memorandum is one of a series being prepared for the Stafford County Water and Sewer Master Plan project. The purpose of this technical memorandum is to present the water demand forecasts for the DOU service area through buildout (2050). Forecasts in this report represent long-term annual average water use and do not reflect seasonal or short-term peak demands. These forecasts have been prepared using the best available data and professional judgement. It is intended that the water demand forecasts presented in this technical memorandum be used as the planning basis for capital improvements for water supply, treatment, and distribution as well as wastewater collection and treatment.

This document contains the following sections:

Terminology, Definitions and Glossary

Average Dry Weather Flow (ADWF) – ADWF consists of average daily sewage flows and groundwater infiltration (GWI). ADWF is the average flow that occurs on a daily basis with no evident reaction to rainfall.

***C-factor* –** A measure of the interior roughness of a pipe**.**

Diurnal Demand or Flow – Fluctuation of water demands or wastewater flows over a 24-hour period.

Effective Storage – Effective storage for each storage facility is determined by establishing the level in each tank above which all points in the water system can be served at 20 psi or higher (based on peak hour or maximum day plus fire flow).

Equalization Storage – The storage of peaking flows to prevent overflows from the sewer collection and conveyance systems.

Groundwater Infiltration (GWI) – Groundwater that infiltrates pipeline and manhole defects located below the ground surface. Groundwater infiltration is separate and distinguished from inflow resulting from storm events. Infiltration is a steady 24-hour flow that usually varies during the year in relation to the groundwater levels above the sewers. Infiltration rates are normally estimated from wastewater flows measured in the sewers during the early morning hours when water use is at a minimum and the flow is essentially infiltration.

H2OMAP Water – H2OMAP Water is a computer model used for modeling the Department of Utilities' water system under various demand conditions.

H2OMAP Sewer – H2OMAP Sewer is a computer model used for modeling the Department of Utilities' sewer system under various flow conditions.

Inflow – Drainage that enters the collection system through illegal or permitted connections, such as catch basins, downspouts, area drains and manhole covers. Inflow is separate and distinguished from infiltration. The inflow rate can be determined from the flow hydrographs recorded with flow meters by subtracting the normal dry weather flow and the infiltration from the measured flowrate.

Infiltration/Inflow (I/I) – The wastewater component caused by rainfall-dependent infiltration/inflow (RDI/I) and groundwater infiltration (GWI).

***Maximum Day Demand* –** The one day in the year when the consumption is the highest.

***Maximum Hour Demand* –** The one hour in the year when water consumption is the highest.

Node – A junction of two or more pipes, commonly representing a point where pipe characteristics change.

Peak Dry Weather Flow (PDWF) – PDWF consists of peak sewage flows plus GWI. PDWF is the highest measured hourly flow that occurs on a dry weather day.

Peak Wet Weather Flow (PWWF) – PWWF consists of ADWF plus RDI/I. PWWF is the highest measured hourly flow that occurs during wet weather.

Peak Factor – Peak factor is PWWF/ADWF.

Pressure Reducing Valve (PRV) – A valve that will maintain a specified downstream pressure.

Pressure Zone – A network of water pipes having a common static hydraulic grade line. Pressure zones are separated by closed valves, pressure regulating valves, pumping stations, and reservoirs.

Rainfall-Dependent Infiltration/Inflow (RDI/I) – RDI/I consists of rainfall that enters the collection system through direct connections (roof leaders, manholes, etc.) and causes an almost immediate increase in wastewater flow.

Service Area – The area served by the water distribution or wastewater collection system.

Steady State Simulation – A network model solution for a single point in time.

Tributary Area – The tributary area of a sewage system consists of all areas that contribute flow to the sewer by gravity and/or force main discharges.

ADD	Average Day Demand
ADWF	Average Dry Weather Flow
AWWA	American Water Works Association
CIP	Capital Improvement Program
cfs	Cubic Feet per Second
CMOM	Capacity, Management, Operation and Maintenance
CWA	Clean Water Act
DOU	Stafford County Department of Utilities
D/DBP	Disinfectants/Disinfection Byproducts
EA	Environmental Assessment
EIS	Environmental Impact Statement
EPA	US Environmental Protection Agency
EPS	Extended Period Simulation
ft	Feet
FY	Fiscal Year
gpcpd	Gallons per Capita per Day
gpd	Gallons per Day
gpm	Gallons per Minute
gpdidm	Gallons per Day per Inch Diameter – Mile
GWI	Groundwater Infiltration
HAAs	Haloacetic Acids
HGL	Hydraulic Grade Line
ICR	Information Collection Rule
I/I	Infiltration and Inflow
IESWTR	Interim Enhanced Surface Water Treatment Rule
ISO	Insurance Service Organization
L	Liter
MCL	Maximum Contaminant Level
MDD	Maximum Day Demand

MG	Million Gallons
MGD	Million Gallons Per Day
mg/l	Milligrams per Liter
mgd	Million Gallons per Day
MPN/100 ml	Most Probable Number per 100 Milliliters
NEPA	National Environmental Policy Act
O&M	Operations and Maintenance
PDWF	Peak Dry Weather Flow
PHD	Peak Hour Demand
PRV	Pressure Reducing Valve
psi	Pounds per Square Inch
PSV	Pressure Sustaining Valve
PWWF	Peak Wet Weather Flow
PWS	Public Water Supply
RDI/I	Rainfall-Dependent Infiltration/Inflow
SCADA	Supervisory Control and Data Acquisition
SDWA	Safe Drinking Water Act
SSO	Sanitary Sewer Overflows
SWTR	Surface Water Treatment Rule
TCR	Total Coliform Rule
THMs	Trihalomethanes
UFW	Unaccounted-for Water
ug/L	Micrograms per Liter
USACE	US Army Corps of Engineers
USEPA	US Environmental Protection Agency
USGS	US Geological Survey
VDEQ	Virginia Department of Environmental Quality
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant

Executive Summary

Water demand projections were developed for each pressure zone under the future buildout condition. Long-term water demand projections from the Rocky Pen Run Reservoir permitting project were used as the basis for buildout (2050) water demands. Land use information and water duties (gallons per day per acre) were used to define how water demands were allocated to the various land use categories throughout the County. The Rocky Pen Run Reservoir data indicate that the projected increase from the water demand of 8.4 mgd (2003) to 27.7 mgd over the next 47 years would represent an annual average increase of 0.41 mgd, which is consistent with the demand increase DOU has historically seen. For this Water and Sewer Master Plan, the average day water demand at buildout (2050) is projected to be 30.8 mgd (slightly higher than the 27.7 mgd identified in the Rocky Pen Run Reservoir permitting project).

Water and sewer utilities have traditionally adopted a conservative approach when planning and sizing facilities with high capital costs and long lead times required for planning, permitting, design and construction. This approach typically includes diligent efforts to avoid underestimating the level of future demands that those facilities will serve. Within this context, it is important to include allowances for the wide range of unknowns inherent in long-range forecasts.

The approach outlined in this technical memorandum gives reasonable projections of future water demands and allows DOU to build conservatism into the sizing of piping in the latter stages of the planning process, thereby minimizing the amount of rework required to update plans and projected improvement projects.

1.0. Description of Service Area

Stafford County is located approximately 40 miles south of Washington, D.C. and 60 miles north of Richmond, Virginia. The County covers 277 square miles of which 51 square miles in the northern portion of the County comprise the Quantico Marine Corps Base. With its proximity to major industrial and commercial markets and its high percentage of undeveloped land, the County is experiencing rapid residential and commercial development. The number of water/sewer accounts has increased from 6,000 in 1982 to over 28,000 in 2004. Currently, the public utility customer base is increasing at an annual rate of 5%. The Stafford County Board of Supervisors has adopted a goal of an annual population increase of 2%.

2.0. Water Demand Forecasting Methods

Nearly all techniques and approaches for projecting future water demands are based on the premise that an analysis of historic trends can serve as the basis for predicting future trends. Annual increases in total water demands for the DOU service area have followed a consistent pattern of growth. These trends provide a strong basis for predicting future water demands for the DOU service area. The three most commonly used methods for applying historic trends as a means for predicting future demands include:

- Extrapolation of Historic Demand
- Per Capita Demand Forecasting
- Disaggregated Demand Forecasting

2.1. Extrapolation of Historic Demands

The extrapolation method is used by many utilities to conduct short-term water demand forecasts of three to five years, but few use this technique for long-term forecasting. The extrapolation method is typically used to assess the overall operational, facility and financial implications of observed trends.

2.2. Per Capita Demand Forecasting

The per capita method is similar to the extrapolation method in that all water users are grouped together. The per capita method, however, links future water uses directly to a projection of population growth. For many water utilities, the per capita method is the long-range forecasting method of choice and is used to accomplish the following:

- Prepare detailed and comprehensive capital improvement project budgets, facility plans, and implementation schedules.
- Conduct detailed and comprehensive cost-benefit analyses, alternative evaluations, and financial and economic analyses.
- Negotiate regulatory compliance requirements and schedules.

2.3. Disaggregated Demand Forecasting

The disaggregated demand method separates (disaggregates) the water demands of a utility into more uniform groups of users as the basis for future projections. This method provides greater accuracy and flexibility in analyzing alternatives because of the ability to use different consumption rates within each sector and different growth rates among sectors. This approach can be used with land use information and water duties (gallons per day per acre) to generate water demands.

2.4. Uncertainty Considerations

Water and sewer utilities have traditionally adopted a conservative approach when planning and sizing facilities with high capital costs and long lead times required for planning, permitting, design and construction. This approach typically includes diligent efforts to avoid underestimating the level of future demands that those facilities will serve. Within this context, it is important to include allowances for the wide range of unknowns inherent in long-range forecasts.

2.5. Recommended Forecasting Approach

For this study, the disaggregated demand forecasting approach is used for projecting future demands because it provides greater accuracy than the other approaches and flexibility in analyzing alternatives. The accuracy and flexibility of this method are based on the ability to use different consumption rates within each sector and different growth rates among sectors.

3.0. Per Capita Water Demands and Water Duties

In terms of the total quantity of water required, water demands are usually estimated on the basis of per capita demand. Variations in water use depend on size of community, geographic location, climate, season, day of week, time of day, and the extent of industrialization. Because of these variations, the only reliable way to estimate future water demands is to study each community separately. To define how the total water use is distributed within a community throughout the day, the best indicator is land use. Table 1 compares the per capita water demands and the water duties (gpd/acre) for various types of regional land use.

Table 1. Per capita water demands and water duties

Reference	Stafford County (Master Plan criteria)[1]	Stafford County (1990 Master Plan)[2]	Anne Arundel County, MD	Loudoun County Sanitation Authority, VA
Per Capita Demand Factor (GPD)	80	70	100	100
Suburban Residential (GPD/Ac)	500	672	471	1000
Urban Residential (GPD/Ac)	1300	1750	1881	1750
Rural Residential (GPD/Ac)	80	80	54	45
Agricultural (GPD/Ac)	40	80	54	45
Commercial (GPD/Ac)	750	1000	1300	Not Defined.
Office (GPD/Ac)	500	2000	1300	Not Defined.
Light Industrial (GPD/Ac)	500	1500	500	Not Defined.
Heavy Industrial (GPD/Ac)	2000	4500	1000	Not Defined.
Institutional (GPD/Ac)	500	1000	1300	Not Defined.

[1] Water duties (gpd/acre) used in the Master Plan were based on a comparison with other utilities, data compiled by the DOU, and the projected growth identified in the water demand projections for the Rocky Pen Run Reservoir project. Note that the water duties in the Master Plan are to be used with 100% development of the land use category (i.e., the gross area includes the area required for existing and future road corridors, on-site stormwater facilities, on-site open space, etc.).

[2] Buildout demands and water duties are used to compute the total water demand which is then reduced to reflect 80% development of the Land Use Plan (i.e., total demand multiplied by 80%). Consequently, the computed water demands and water duties in Table 2 for "Stafford County 1990 Master Plan" will ultimately be lower than shown to reflect the anticipated acreage of actual development.

As shown in Table 1, the per capita water demands and water duties used for this Master Plan are generally lower than data used in DOU's 1990 Master Plan and those used by neighboring utilities. Many utilities apply a global reduction factor (typically 70-90%) after the total water demand is computed to reflect the reduction in the level of development of the land use category (i.e., the gross area which includes the area required for existing and future road corridors, on-site stormwater facilities, on-site open space, etc.). Rather than apply a global reduction factor after computing the total water demand, the water demands and water duties were reduced for each land use category prior to compiling the water demands.

4.0. Application of Water Demand Forecasting Methods

4.1. Projected Water Demands for Buildout Conditions

A detailed water demand forecast was recently developed in support of the DOU's proposed Rocky Pen Run Reservoir permitting project and the buildout water demands in this study are based on the Rocky Pen Run Reservoir water demand forecast. The objective of the demand analysis for this Master Plan was to determine where the water demands should be allocated throughout the County. This was accomplished by developing an independent water demand projection based on the most recent Land Use information and revising the computed demands as needed to match the projections generated for the Rocky Pen Run Reservoir project.

Using the disaggregated demand forecasting approach, water demand projections were developed using GIS layers provided by DOU and the County's Planning Department. The critical layers that were used in the analysis included:

- Pressure zones – This layer delineates the five existing water system pressure zone boundaries within the County. Pressure zones for the buildout condition were delineated for the entire County (except for Quantico Marine Corps Base) based on topographic data obtained from the County and maximum and minimum water system pressure criteria. Water service can be extended to the entire County and service outside the limits of the Urban Service Area is allowed, but is generally limited to groundwater well failures.

- Land use – This layer identifies 16 alternative land use designations throughout the County (i.e., residential, commercial, etc.).

After the shapefiles were obtained from the County, the following general methodology was used to estimate the future water demands:

- Compute the acreage for each land use category in the County.
- Apply water duties (gpd/acre) for each land use type.
- Add the projected Federal or Military (FED) demand (1.5 mgd).
- Add the unaccounted-for water (UAW) portion of the total demand (15%).
- Subtract the conservation component of the total demand (8%).

A detailed description of these steps follows.

1. The first step was to develop a layer containing only the developable land within the County. In order not to overestimate the projection of future water demands, it is necessary to subtract the "undevelopable" land use from the Land Use Plan. Undevelopable land included environmentally sensitive areas (Resource Protection Areas) and Parks (PRK) which were both excluded from the developable land estimate. This was accomplished by using the Land Use shapefile and querying out the PRK and RPA areas. It is also appropriate to recognize that full development of the Land Use Plan at the allowable densities is not likely due to the retention of some large tracts that will not be subdivided to the full development density, space for on-site stormwater facilities, on-site open space for buffers, space for existing and future road corridors, etc. The criteria which were used in the previous Master Plan reduced the computed water demands to reflect 80% development of the Land Use Plan as the buildout condition. This approach was not used in this study. Rather, water duties for various land use categories were held below the typical levels to reflect actual conditions. In addition, it was not possible to readily distinguish whether the developable land was currently vacant or non-vacant. Rather than attempt to identify the amount of vacant land remaining to be developed (i.e., amount of infill), future water demands were projected for both the vacant and non-vacant portions of the developable land as though the entire area was vacant. This approach for calculating water demands from developable land use data provides reasonable demand projections for long-range master planning.
2. After the developable land layer was generated, it was unioned with the water system pressure zones to form a new composite layer. Through this process of unioning the layers, polygons were created based on the intersections of the boundaries of the areas (land use and pressure zones).
3. Land use categories used in County's shapefiles (2003) were grouped into the categories used in Table 2-17 from the Rocky Pen Run Reservoir project (Column A of Table 2). Table 3 presents the data from Table 2-17 of the Rocky Pen Run Reservoir project.

4. Water demand data from Table 2-17 of the Rocky Pen Run Reservoir project were entered into Column F of Table 2. These values were fixed.
5. Acreage calculated from the County's land use shapefiles for each land use category was entered in Column C of Table 2. These values were fixed. It should be noted that the acreage of land use shown in Table 2 represents the entire County (except for Quantico Marine Corps Base). Water service can be extended to the entire County, but is generally limited to groundwater well failures. Including the water demands for the rural residential and agricultural areas of the County allows DOU to make provisions, if desired, in the centrally located pumping and transmission facilities needed to serve these areas.
6. After the composite layer was created, water duties (gpd/acre) were added to the composite layer database using the land use classification (e.g., SRE for Suburban Residential = 500 gpd/acre, etc.). Water duties traditionally used by DOU were entered to initially compute the water demand for each category in Column E of Table 2.
7. Water duties were iteratively (and relatively proportionately) adjusted downward until the Computed Water and Sewer Master Plan Demand (Column E of Table 2) matched the demands presented in Table 2-17 of the Rocky Pen Run Reservoir project (Column F of Table 2). The computed water duties were then checked for reasonableness.
8. The percentage of unaccounted-for water and water conservation were taken from the Rocky Pen Run Reservoir study.
9. The area for RRE was not included in the Rocky Pen Run Reservoir study and essentially accounts for the differences shown for the "Residential" and "Total Demand" values in Table 2.

After computing the water demands for each land use, the demands were allocated to the model nodes using the H2OMAP Water model as described in Technical Memorandum 4.

Table 2. Water demands for buildout conditions

Column A	Column B	Column C	Column D	Column E	Column F	Column G
Land Use	**Proposed Water Duty (gpd/acre)**	**Computed Area from April 2003 Land Use (acres)**	**Computed Water and Sewer Master Plan Demand (mgd)**	**Computed Water and Sewer Master Plan Demand (mgd)**	**Table 2-17 Rocky Pen Run Water Demand (mgd)**	**Table 2-17 Rocky Pen Run Water Use Category**
Residential				16.8	14.0	Residential
Suburban Residential (SRE)	500	19,427	9.71			
Urban Residential (URE)	1,300	1,887	2.45			
Rural Residential (RRE)	80	35,424	2.83			
Agricultural (AGR)	40	45,768	1.83			
Commercial/Institutional/Light Industrial				10.2	10.2	- Commercial - Institutional - Light Industrial - Heavy Industrial
Commercial (UCM, SCM, RCM)/Neighborhood Center (NCT)	750	4,915	3.69			
Office (OFF)	500	201	0.10			
Light Industrial (LIN)/Business (BUS)	500	10,529	5.26			
Institutional (INS)	500	1,710	0.86			
Heavy Industrial (HIN)	2,000	127	0.25			
Federal or Military (FED)				1.5	1.5	Military
Subtotal Demand [1]				28.5	25.6	
Unaccounted-for Water (15% of Total Demand)				5.0	4.5	
Total Demand (without Additional Conservation)				33.5	30.1	
Additional Conservation (8% of Total Demand)				2.7	2.4	
Total Demand (with Additional Conservation)				**30.8**	**27.7**	

[1] Rounding-off error for subtotal demand from Table 2-17. Commercial/Institutional/Light Industrial demand of 10.15 mgd was developed in the Rocky Pen Run Reservoir study and rounded off to 10.2 mgd in Table 2-17 of Rocky Pen Run study.

Table 3. Summary of projected Stafford County potable water demand (Table 2-17 from Rocky Pen Run Reservoir study)

Category	**2000**	**2010**	**2020**	**2030**	**2040**	**2050**
Total County Population	92,446	123,998	149,994	175,990	201,986	227,982
Residential						
Population Served	69,335	96,718	121,495	147,832	175,728	205,184
Per Capita Use (gpd)	68	68	68	68	68	68
Demand (mgd)	4.7	6.6	8.3	10.1	11.9	14.0
Commercial/Institutional/Light Industry						
Commercial Employment	23,112	35,959	49,498	65,116	82,814	102,592
Per Employee Use (gpd)	45	48	51	54	57	60
Demand (mgd)	1.0	1.7	2.5	3.5	4.7	6.2
Heavy Industry						
Demand (mgd)	0.0	0.8	1.6	2.4	3.2	4.0
Military						
Water Sales (mgd)	0.75	1.25	1.5	1.5	1.5	1.5
Subtotal Demand (mgd)	6.5	10.4	13.9	17.5	21.4	25.6
Unaccounted-for Water (UAW)						
% of Total Demand	20	17.5	15	15	15	15
Demand (mgd)	1.6	2.2	2.5	3.1	3.8	4.5
Total Demand (mgd)						
Without Additional Conservation	8.1	12.5	16.3	20.6	25.1	30.1
Future Water Savings (%)	0	2.7	5.3	8	8	8
With Additional Conservation (mgd)	**8.1**	**12.2**	**15.5**	**18.9**	**23.1**	**27.7**

4.2. Historic Water Losses

It is important to note that the total "water sold" to DOU customers, or water measured at water meters, is typically 15 percent less than the "water produced" at the existing Smith Lake and Abel Lake WTPs. This is due to normal consumptive losses in the water distribution system. Losses in the water distribution system are typically labeled *unaccounted-for water*.

Unaccounted-for water is a term commonly used in the water industry. Unfortunately, it is used by different utilities with significant differences in definition, making comparison of numbers that are reported by different agencies difficult unless specific inquiries are made as to what is included in each system's report. Based on discussions with DOU, "unaccounted-for water" as defined by DOU can be referred to as unmetered (unbilled) water which is calculated as the difference between water produced by DOU and that metered (billed) to DOU's retail customers. There are no specific industry accepted standards for unaccounted-for water, however, the American Water Works Association's Leak Detection and Accountability Committee has recommended a goal of less than 10% (Journal AWWA, July 1996). (It should be noted that the procedure described by the AWWA Committee allows for subtraction of losses that have been specifically identified and quantified, and therefore "accounted-for"). DOU's water demand projections include an unaccounted-for water allowance of 15% of the total demand.

The unaccounted-for water allowance was factored into the water demands as a global demand increase of 15% in the water model based on work developed in the Rocky Pen Run Reservoir permitting project.

4.3. Water Conservation

For this study, water conservation at the buildout condition was factored in as a global water demand reduction of 8% in the water model. This reduction was taken from the work developed for the Rocky Pen Run Reservoir project.

4.4. Projected Water Demands for Near-term Conditions

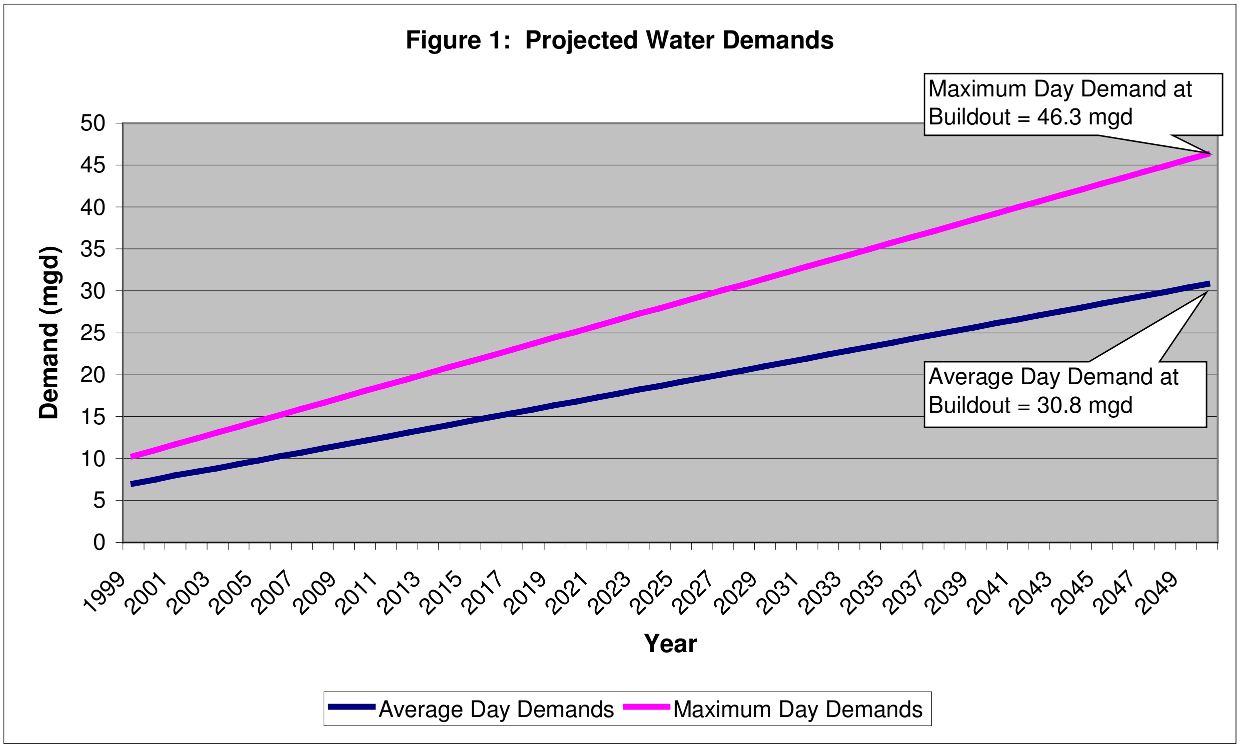
In order to identify water system improvements needed through roughly 2013 when Rocky Pen Run Reservoir is expected to be on-line, maximum day water demands at buildout (46.3 mgd) were reduced uniformly to a maximum day demand of 20 mgd which matches the production capacity for Smith Lake and Abel Lake WTPs. The objective of this analysis was to identify what facilities are needed and the size of those facilities to convey water from the Smith Lake and Abel Lake WTPs until the Rocky Pen Run Reservoir WTP is on-line.

4.5. Peaking Factors

Water systems are required to supply flow at rates that fluctuate over a wide range from day-to-day and hour-to-hour. Rates most important to planning, design and operation of a water system are average day, maximum (peak) day, maximum (peak) hour, and maximum hour plus fire flow.

- <u>Average day demand</u> is the total volume of water delivered to the system in a given year divided by the number of days in the year.
- <u>Maximum (peak) day demand</u> is the largest quantity of water supplied to the system on any given day of the year.
- <u>Maximum (peak) hour demand</u> is the highest rate of flow for any hour in a year.
- <u>Maximum day plus fire flow</u> considers the possibility of a fire event under maximum day demand conditions.

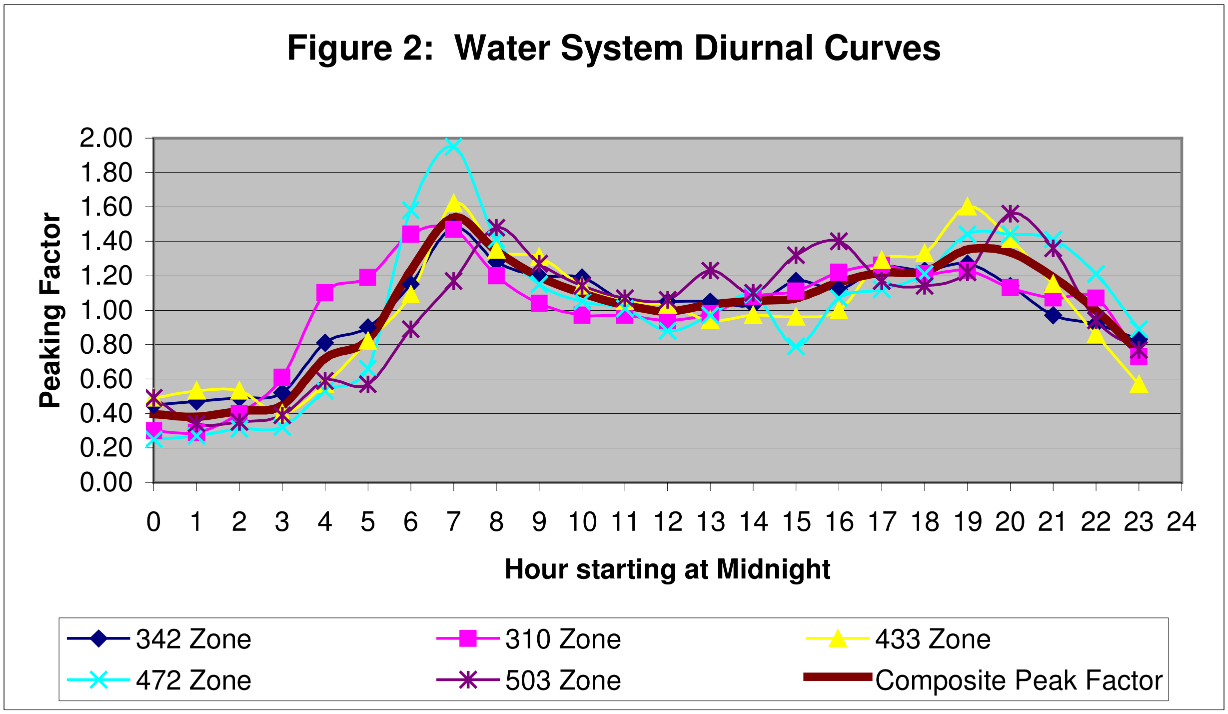
The peak day factor (maximum day demand / average day demand) for 2002 was 1.67. Peaking factors will drop as the system continues to expand through the planning period. Average water demands are expected to increase from 8.4 mgd (2003) to roughly 30.8 mgd under buildout (2050) conditions. During the same period, the maximum day demands are expected to increase from approximately 13 mgd (2003) to 46 mgd at buildout (2050) based on a peaking factor of 1.5 times the average day demand. The water demands are shown in Figure 1.



4.6. Diurnal Curves

Diurnal curves are simply representations of how water is used over time of day. Diurnal curves are different for each house, each industry and each water user. However, for the purpose of creating a model to represent a water distribution system, simplifications are generally made such that all residential, commercial, industrial, and other water use classifications are each assumed to have consistent water demand (diurnal) curves.

Water demands vary throughout the day with peaks in the morning and evening and low flows in the early morning hours. Patterns are used to represent the daily temporal variations within the water system. They consist of a collection of multipliers (multiplication factors) that are applied to the average day demand to allow it to vary over time during an extended period simulation (EPS). Different patterns can be applied to individual water nodes or groups of nodes to accurately represent water use categories (e.g., residential, commercial, etc.). For this Master Plan, the diurnal data provided by DOU was used to calibrate the water model and conduct the modeling analyses. The diurnal demand patterns are shown in Figure 2 and were used for each pressure zone. Consequently, the average demand at each water node was multiplied by the diurnal demand pattern for the pressure zone to predict the water use throughout the day.



5.0. Findings and Recommendations

Water demands were computed for each pressure zone for the future buildout and near-term conditions. Future pressure zone boundaries were established or modified using topographic data provided by DOU and maximum and minimum pressure criteria. The projected increase from the average day water demand of 8.4 mgd (2003) to 30.8 mgd over the next 47 years would represent an annual average increase of 0.47 mgd, which is consistent with the demand increase DOU has historically seen. For the Water and Sewer Master Plan, the average day water demand at buildout (2050) is projected to be 30.8 mgd and the maximum day demand for the near-term condition (2013) is 20 mgd to match the production capability of the water treatment plants.

The approach outlined in this technical memorandum gives reasonable projections of future water demands and allows DOU to build conservatism into the sizing of piping in the latter stages of the planning process, thereby minimizing the amount of rework required to update plans and projected improvement projects.

TECHNICAL MEMORANDUM 3

Water Treatment Plant Siting Evaluation

Prepared for: Stafford County Department of Utilities
Prepare by: O'Brien & Gere
Date: November 2004

This technical memorandum is one of a series being prepared for the Stafford County Water and Sewer Master Plan project. The purpose of this memorandum is to identify a location for treating the raw water from the proposed Rocky Pen Run Reservoir that provides system reliability and flexibility while allowing water to be delivered cost-effectively where and when it is needed. To identify this location, a site comparison matrix was developed that facilitates comparison of alternative locations against a number of pre-defined evaluation criteria. After developing the site comparison matrix, meetings were held with Stafford County on May 5 and May 22, 2003 to refine the matrix, define the evaluation criteria, obtain consensus that the correct locations were being investigated, and review the preliminary analysis.

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Maximum Hour Demand – The one hour in the year when water consumption is the highest.

Node – A junction of two or more pipes, commonly representing a point where pipe characteristics change.

Peak Dry Weather Flow (PDWF) – PDWF consists of peak sewage flows plus GWI. PDWF is the highest measured hourly flow that occurs on a dry weather day.

Peak Wet Weather Flow (PWWF) – PWWF consists of ADWF plus RDI/I. PWWF is the highest measured hourly flow that occurs during wet weather.

Peak Factor – Peak factor is PWWF/ADWF.

Pressure Reducing Valve (PRV) – A valve that will maintain a specified downstream pressure.

Pressure Zone – A network of water pipes having a common static hydraulic grade line. Pressure zones are separated by closed valves, pressure regulating valves, pumping stations, and reservoirs.

Rainfall-Dependent Infiltration/Inflow (RDI/I) – RDI/I consists of rainfall that enters the collection system through direct connections (roof leaders, manholes, etc.) and causes an almost immediate increase in wastewater flow.

Service Area – The area served by the water distribution or wastewater collection system.

Steady State Simulation – A network model solution for a single point in time.

Tributary Area – The tributary area of a sewage system consists of all areas that contribute flow to the sewer by gravity and/or force main discharges.

ADD	Average Day Demand
ADWF	Average Dry Weather Flow
AWWA	American Water Works Association
CIP	Capital Improvement Program
cfs	Cubic Feet per Second
CMOM	Capacity, Management, Operation and Maintenance
CWA	Clean Water Act
DOU	Stafford County Department of Utilities
D/DBP	Disinfectants/Disinfection Byproducts
EA	Environmental Assessment
EIS	Environmental Impact Statement
EPA	US Environmental Protection Agency
EPS	Extended Period Simulation
ft	Feet
FY	Fiscal Year
gpcpd	Gallons per Capita per Day
gpd	Gallons per Day
gpm	Gallons per Minute
gpdidm	Gallons per Day per Inch Diameter – Mile
GWI	Groundwater Infiltration
HAAs	Haloacetic Acids
HGL	Hydraulic Grade Line
ICR	Information Collection Rule
I/I	Infiltration and Inflow
IESWTR	Interim Enhanced Surface Water Treatment Rule
ISO	Insurance Service Organization
L	Liter
MCL	Maximum Contaminant Level
MDD	Maximum Day Demand

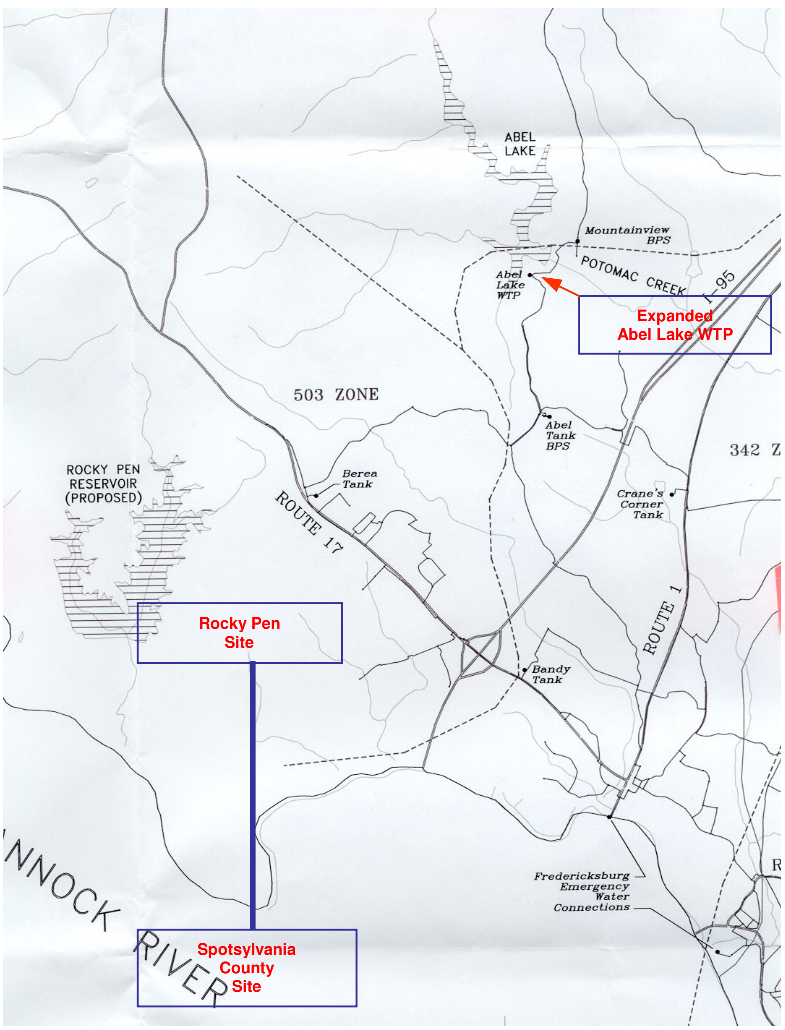
MG	Million Gallons
MGD	Million Gallons Per Day
mg/l	Milligrams per Liter
mgd	Million Gallons per Day
MPN/100 ml	Most Probable Number per 100 Milliliters
NEPA	National Environmental Policy Act
O&M	Operations and Maintenance
PDWF	Peak Dry Weather Flow
PHD	Peak Hour Demand
PRV	Pressure Reducing Valve
psi	Pounds per Square Inch
PSV	Pressure Sustaining Valve
PWWF	Peak Wet Weather Flow
PWS	Public Water Supply
RDI/I	Rainfall-Dependent Infiltration/Inflow
SCADA	Supervisory Control and Data Acquisition
SDWA	Safe Drinking Water Act
SSO	Sanitary Sewer Overflows
SWTR	Surface Water Treatment Rule
TCR	Total Coliform Rule
THMs	Trihalomethanes
UFW	Unaccounted-for Water
ug/L	Micrograms per Liter
USACE	US Army Corps of Engineers
USEPA	US Environmental Protection Agency
USGS	US Geological Survey
VDEQ	Virginia Department of Environmental Quality
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant

1.0. Purpose and Scope of Services

The purpose of the Water Treatment Plant (WTP) Siting Evaluation is to identify a location for treating the raw water from the Rocky Pen Run Reservoir that provides system reliability and flexibility while allowing water to be delivered cost-effectively where and when it is needed. In order to identify the optimum site for the water treatment plant, estimates of the differential capital and operating costs associated with each alternative treatment plant site were needed. To simplify the analysis of the cost estimates and allow the project team to more readily assess the differences in evaluation criteria between the various site alternatives, a site comparison matrix was created.

2.0. Site Comparison Matrix

On May 5, 2003, a meeting was held with Stafford County to discuss the site comparison matrix, define the evaluation criteria for which capital and operating costs would be estimated, and obtain consensus that the correct locations were being investigated. At this meeting, three alternative sites for construction of a water treatment plant were discussed (see figure below). Subsequent to the meeting, variations of Alternatives #3, #4, and #5 were added to illustrate cost and timing issues.



<u>Alternative #1</u>
Construct a new water treatment plant (WTP) at the Rocky Pen Run Reservoir site to treat the water from that supply. In this situation, the Smith Lake and Abel Lake WTP's would remain in service at their existing capacities.

<u>Alternative #2</u>
Construct new treatment units at the Motts Run WTP in Spotsylvania County to treat the water from Rocky Pen Run Reservoir. In this situation, the Smith Lake and Abel Lake WTP's would remain in service at their existing capacities.

<u>Alternative #3</u>
Construct a new WTP on the existing Abel Lake WTP site (or on parcels adjacent to this site) to treat the water from both Rocky Pen Run Reservoir and Abel Lake. Under Option A, the existing Abel Lake WTP would be kept in service for approximately 20 years, at which point it would be decommissioned and 6 mgd of treatment capacity added to the new, consolidated WTP. Under Option B, all of the planned treatment capacity

at the consolidated facility would be constructed immediately, at the same time as the decommissioning of the existing Abel Lake WTP. The Smith Lake WTP would remain in service at its existing capacity under either option.

<u>Alternative #4</u>
Construct a new WTP adjacent to the Abel Lake Finished Water Storage Tank site to treat the water from both the Rocky Pen Run Reservoir and Abel Lake. The timing options investigated under this alternative are the same as investigated under Alternative #3.

<u>Alternative #5</u>
Construct a new WTP at the Rocky Pen Run Reservoir site to treat the water from both the Rocky Pen Run Reservoir and Abel Lake. The timing options investigated under this alternative are the same as investigated under Alternative #3.

At the May 5, 2003 meeting with DOU, six evaluation criteria were suggested for use in the plant siting study including:

- Water Treatment Plant Costs
- Pumping Station Costs
- Pipeline Costs
- Up-front Capital Expenditure Costs
- Staffing Costs
- Power

Based upon the preliminary analysis and the discussion at the May 22, 2003 progress review meeting, the "Power" criterion was dropped. The remaining criteria used in the evaluation are as follows:

- <u>Water Treatment Plant Costs</u> - The water treatment plant cost criterion was used to evaluate the cost of constructing new, or expanding existing, treatment facilities to treat the raw water from the Rocky Pen Run Reservoir. The required treatment capacity for a new WTP was estimated by applying a peak factor of 1.5 to the estimated year 2050 average day demand of 27.7 mgd, and subtracting the current reliable treatment capacity (14 mgd at the Smith Lake WTP and 6 mgd at the Abel Lake WTP). The capital cost of constructing a new treatment facility was generally based on a unit cost of $1.25/gallon. The cost of constructing additional capacity at the Motts Run WTP in Spotsylvania County was based on a unit cost of $1.67/gallon, which was calculated based on the construction cost for the existing Motts Run WTP.

  For those site alternatives involving the use of the existing Abel Lake WTP, it is understood that various plant upgrades are required to extend the useful life of the facility approximately 20 years (see facility assessment for Abel Lake WTF presented in Appendix A for TM#3). It should be noted, however, that the cost of the upgrades would be included in any site alternative involving the use of this facility and are therefore not true differential costs. For those site alternatives where the existing Abel Lake WTP would be kept in service throughout the entire planning period (rather than decommissioned after 20 years or so in favor of a consolidated facility), a 30-year present worth cost of $650,000 was added for annual maintenance. The estimated 30-year present worth of the annual maintenance cost was based on an annual maintenance allowance of 1 percent of the plant value, or approximately $60,000. The reason this cost was added was that operating a 40 to 70 year-old Abel Lake facility will be more maintenance intensive than operating a new, consolidated facility (the existing Abel Lake WTP will be roughly 70 years old at the end of the planning period). A 30-year present worth was used because the annual maintenance cost would be incurred after the initial capital investments at the facility are made, which are intended to extend the useful life 20 years. Because the study is over a 50-year planning period, the annual maintenance cost would be incurred over 30 years.

For those options where it was assumed that a portion of the total treatment capacity would be deferred, the cost of treatment facility construction was broken down into two components: headworks and treatment units. Assuming the cost of headworks construction would be accounted for separately, the unit cost of treatment process units was estimated to be approximately $0.65/gallon. Using the overall unit cost of $1.25/gallon for a complete WTF, and subtracting the cost for the total planned treatment capacity (at the $0.65/gallon cost), the construction cost for the headworks of a consolidated treatment facility was estimated to be approximately $19 million. In order to account for the deferral of the final 6 mgd of treatment capacity, the construction cost for 6 mgd (based on the treatment process unit cost of $0.65/gal) was converted to a present value assuming a 20-year period and a real discount rate of 3 percent.

- Pumping Station Costs - The pumping station cost criterion was used to evaluate the cost of constructing new pumping stations to convey either raw water to the new or expanded treatment facility or finished water to the relevant portions of the distribution system. For the alternatives under analysis, the construction costs of pumping stations with capacities greater than or equal to 10 mgd were based on a unit cost of $0.15/gallon, while the construction cost of a pumping station with a capacity of 6 mgd (i.e., a pumping station to convey raw water from Abel Lake to a new, consolidated treatment facility) was based on a lump sum cost of $1 million.

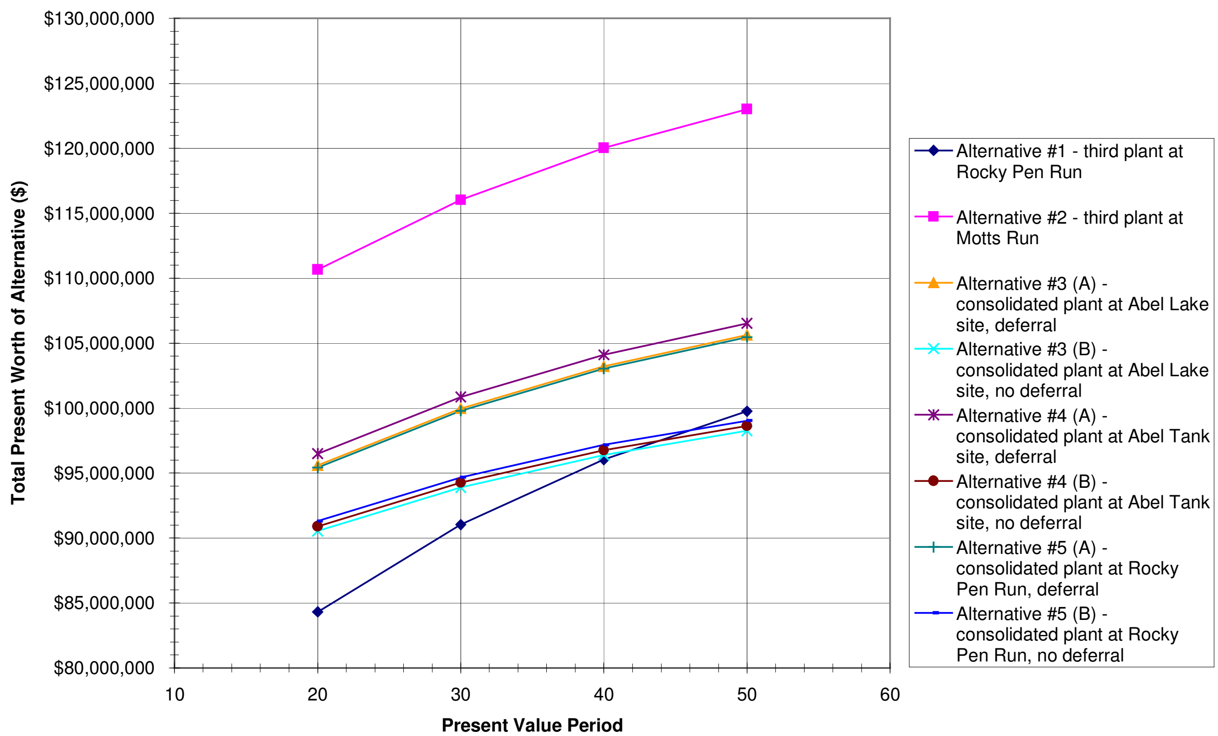
- Pipeline Costs - The pipeline cost criterion was used to evaluate the cost of constructing new pipelines to convey either raw water to the new or expanded treatment facility or finished water to the relevant portions of the distribution system. For the alternatives under analysis, the pipes were generally sized for a velocity of approximately 6 ft/sec at maximum day flows. Additional costs were added to Alternative #2 (Motts Run WTP) for construction of a tunnel below the Rappahannock River to convey raw water to Motts Run from the Rocky Pen Run Reservoir, and to convey finished water back to Stafford County. [Note to reader: At the May 5th meeting, it was suggested that it could be more economical to release raw water from Rocky Pen Run Reservoir to the Rappahannock River and withdraw it at the Motts Run intake. Upon further review, it appears that the intake and raw water system at Motts Run are not sized to allow this, and therefore would require new permits to expand. The cost to tunnel is likely comparable to, and more readily permitted, than expanding the Motts Run intake and has been used for this analysis.]

- Up-front Capital Expenditure Costs - The up-front capital cost criterion was used to capture other capital costs that would be expended as part of the implementation of the selected treatment facility alternative. The first component of this criterion was land acquisition costs, which includes the cost to purchase the land on which the treatment facility would be located. The cost of the land was assumed to be equal to the assessed value of the property. The second component of this criterion was site development costs, which could include the costs for site grading, roadwork, etc. Because these components are site specific, costs for site development measures were estimated after the preliminary screening step.

- Staffing Cost - The staffing cost criterion was used to evaluate the annual operating cost associated with the staff required to operate the new or expanded water treatment facility. As reported to O'Brien & Gere at the May 5 meeting, Stafford County currently employs 10 staff (one plant manager, 8 operators, and one mechanic) at each water treatment facility. It was assumed that a new water treatment facility would require the addition of 10 staff to the payroll. The annual operating cost of an additional employee was calculated by averaging the plant payroll of $711,394 ($459,483 in salary, $66,272 in overtime, and $185,639 in benefits) over 10 staff. This calculation resulted in a unit cost per employee of $71,139.

For each site alternative, the estimated capital costs that were input into the site comparison matrix for the water treatment facility, pumping station, pipeline, and up-front capital expenditure criteria were summed and 20 percent markups applied for both contingency and engineering/legal/administration to develop an overall "project cost". The unit cost of constructing the treatment facility (in $/gallon) was then calculated (using the actual gallons/day of treatment capacity constructed) to provide a basis for comparing the project cost of each alternative location. The annual operating costs for the staffing and power criteria were converted to present values using periods of 20 to 50 years and a real discount rate of 3 percent. Finally, the total present value of each alternative was calculated by adding the total project cost of each alternative to the total present value of the annual operation and maintenance cost. Figure 1 shows the full range of 20-year to 50-year present worths.

Figure 1. Effect of Present Value Period on Total Present Worth of Alternatives



Estimates of the capital and operating costs for each facility location alternative are attached to this memorandum.

3.0. Key Assumptions

Several assumptions were made in order to complete the site comparison matrix. These key assumptions and their impacts are summarized below.

- Backwash. For the alternative where a consolidated treatment facility would be located adjacent to the Abel Lake Finished Water Storage Tank site, a new backwash handling system would need to be developed. Since the current practice of returning the backwash water to Abel Lake could not be continued at this remote location, it was assumed that a membrane treatment unit, capable of treating 5% of the total consolidated plant flow, would be constructed to allow the permeate to be returned to the finished water flow. The treatment process capacity was therefore de-rated by 5% when the construction cost was calculated, with the cost of membranes added (assuming a unit cost of $2/gallon and a capacity of 5% of total plant flow).

- Raw Water Pumping. It was assumed, regardless of whether the Rocky Pen Run Reservoir water would be treated on-site or at a remote treatment facility, that a raw water pumping station would be needed at Rocky Pen Run Reservoir.

- Raw Water Conveyance. To estimate the piping costs for the Motts Run WTP site alternative, it was assumed that the raw water supply from Rocky Pen Run Reservoir to Motts Run would be hard-piped, as opposed to gravity flow from Rocky Pen Run Reservoir to an intake along the Rappahannock River. While the raw water pipe, as well as the finished water pipe back to Stafford County, would require a tunnel beneath the Rappahannock River, this added cost is likely comparable to the cost to expand the river intake and convey flow to the WTP.

- Land. Based on information provided by Stafford County, it was assumed that Spotsylvania County owns adequate land at the Motts Run WTP site to construct an additional 21.6 mgd, and a land value equal to the Rocky Pen Run Reservoir site was included.

- Drinking Water Treatment Regulations. Because the water quality is similar among the various sources, it was assumed that future drinking water treatment regulations would affect all potential sites equally. Therefore, no differential costs were included for regulatory compliance.

- Staffing. It was assumed that the Motts Run WTP alternative or the consolidated treatment facility alternatives (with partial treatment capacity deferral) would require the addition of 3 employees to the current plant staff (one assistant plant manager and two operators/mechanics). In the case of the Motts Run WTP, it was assumed that Stafford County would bear the cost of roughly half of that facility's total staff (assumed to be 6 employees). The reasoning behind the assumption is that, while only 3 additional employees were added to the current plant staff, Stafford County would account for roughly half of the total treatment capacity at the Motts Run plant. Therefore, they will likely be responsible for the cost of the staff required to run half of the plant. In the case of a consolidated treatment facility in Stafford County, it was assumed that the differential cost of the alternative would only be the added staff (because Stafford County already bears the cost of the existing staff for County treatment facilities).

- Residuals Disposal. It was assumed that the cost of residuals disposal (i.e., sewer disposal, land application or mechanical dewatering) would be the same at each water treatment facility site, and, therefore, it is not included in the analysis.

4.0. Results

The primary advantages and disadvantages of each treatment facility location alternative, as identified through the siting study, are summarized in Table 1. For purposes of comparison, Alternative #1 was selected as the "baseline" option. As discussed previously in this memorandum, under this alternative the existing Abel Lake and Smith Lake Water Treatment Facilities (WTFs) would continue to treat raw water from their respective supplies at their current permitted treatment rates. A new WTF would be constructed at the Rocky Pen Run Reservoir to treat the raw water from that supply.

Table 1. Summary of water treatment facility siting evaluation results

Alternative	Advantages	Disadvantages
Alternative #1 – Utilize Smith Lake, Abel Lake and Rocky Pen Run Reservoir	• Provides system redundancy • Requires the least capital expenditure of all alternatives • Lowest present worth cost through 40 year analysis • Provides security at each raw water supply through constant presence at the site • Raw water is treated close to the demand centers	• Requires most staff of all alternatives
Alternative #2 – Utilize Smith Lake, Abel Lake, and Motts Run	• Provides system redundancy • Less staff required than under Alternative #1	• Additional capital expense associated with the tunnel beneath the Rappahannock River to convey raw / finished water between Rocky Pen Run Reservoir and Motts Run • Requires additional power to pump raw water to Motts Run and finished water back to Stafford County
Alternative #3 (A) – Utilize a consolidated treatment facility at the ex. Abel Lake WTF site, deferral	• Less staff required than under Alternative #1	• Additional capital expense associated with the long length of 36" pipe to convey raw water from Rocky Pen Run Reservoir to Abel Lake • Less system redundancy than in Alternatives #1 and #2 • Reduced security at raw water supplies due to lack of constant presence • Raw water is conveyed away from the demand centers to be treated • Potential difficulty in acquiring developed properties
Alternative #3 (B) – Utilize a consolidated treatment facility at the ex. Abel Lake WTF site, no deferral	• Requires less staff than under both Alternatives #1 and #3 (A)	• Additional capital expense associated with the long length of 36" pipe to convey raw water from Rocky Pen Run Reservoir to Abel Lake • Less system redundancy than in Alternatives #1 and #2 • Reduced security at raw water supplies due to lack of constant presence • Raw water is conveyed away from the demand centers to be treated • Potential difficulty in acquiring developed properties
Alternative #4 (A) – Utilize a consolidated treatment facility at the Abel Lake Tank site, deferral	• Less staff required than under Alternative #1	• Additional capital expense associated with the long length of 36" pipe to convey raw water from Rocky Pen Run Reservoir to Abel Lake • Less system redundancy than in Alternatives #1 and #2 • Reduced security at raw water supplies

Alternative	Advantages	Disadvantages
		due to lack of constant presence • Raw water is conveyed away from the demand centers to be treated
Alternative #4 (B) – Utilize a consolidated treatment facility at the Abel Lake Tank site, no deferral	• Requires less staff than under both Alternatives #1 and #4 (A)	• Additional capital expense associated with the long length of 36" pipe to convey raw water from Rocky Pen Run Reservoir to Abel Lake • Less system redundancy than in Alternatives #1 and #2 • Reduced security at raw water supplies due to lack of constant presence • Raw water is conveyed away from the demand centers for treatment
Alternative #5 (A) – Utilize a consolidated treatment facility at the Rocky Pen Run Reservoir site, deferral	• Less staff required than under Alternative #1	• Additional capital expense associated with the long length of 16" pipe to convey raw water from Abel Lake to Rocky Pen Run • Less system redundancy than in Alternatives #1 and #2 • Reduced security at raw water supplies due to lack of constant presence
Alternative #5 (B) – Utilize a consolidated treatment facility at the Rocky Pen Run Reservoir site, no deferral	• Requires less staff than under both Alternatives #1 and #5 (A)	• Additional capital expense associated with the long length of 16" pipe to convey raw water from Abel Lake to Rocky Pen Run Reservoir • Less system redundancy than in Alternatives #1 and #2 • Reduced security at raw water supplies due to lack of constant presence

Under Alternative #2, raw water from the Rocky Pen Run Reservoir would be conveyed to an expanded Motts Run WTP for treatment. By treating the Rocky Pen Run Reservoir water at a third water treatment plant, the benefit of improved system redundancy is maintained. An additional benefit of sending the raw water to Motts Run is that Stafford County does not have to bear the annual operating cost associated with staffing an entire facility. The primary disadvantage associated with this alternative is that the capital costs for treating water at Motts Run are much higher than the capital costs estimated for treating the raw water at Rocky Pen Run Reservoir. The capital cost of constructing additional treatment capacity at Motts Run is higher than at one of the possible sites in Stafford County (due to the type of facility constructed at Motts Run), and there is also significant cost associated with the long lengths of pipelines required to convey the raw water to, and finished water away from, Motts Run (including the tunnel under the Rappahannock River).

The remaining alternatives assume that a consolidated treatment facility would be constructed somewhere in Stafford County to treat the raw water from both Abel Lake and the Rocky Pen Run Reservoir (Alternatives #3, #4, and #5). For those consolidation alternatives where it was assumed that a portion of the total planned treatment capacity could be deferred (Option "A"), the primary advantage compared to the baseline condition is that a smaller plant staff is required to operate the consolidated facility. Deferral produces capital cost savings when compared to the cost to construct all of the planned treatment capacity at one time. For those options where deferral of portions of the total planned construction was not assumed (Option "B"), additional reductions in plant staff compared to the baseline and Option "A" alternatives are gained by avoiding the need to operate three "separate" plants during the deferral stage.

The primary disadvantages of the consolidated water treatment facility alternatives compared to the baseline alternative are reduced system redundancy, reduced security at raw water supplies due to the lack of a constant presence, and the additional capital expense to convey raw water either from the Rocky Pen Run Reservoir to Abel Lake or from Abel Lake to Rocky Pen Run Reservoir. The added capital expense is more pronounced under the alternatives where the Rocky Pen Run Reservoir raw water is conveyed to the Abel Lake sites due to the larger pipe size required to convey the greater raw water flow (roughly 25.6 mgd from Rocky Pen Run Reservoir to Abel Lake versus 6 mgd if the flow pattern was reversed).

The impact of the above advantages and disadvantages on the total project cost of each alternative was assessed by estimating capital and operating cost for each of the evaluation criteria at the five alternative treatment facility locations (as described under the investigation section). The treatment facility location alternative with the highest total project cost was found to be the Motts Run WTP site (Alternative #2). All other alternatives had relatively similar present worth costs (within 5% on a 50-year basis), which is within the level of accuracy of these estimates. The criteria that appeared to have the greatest influence on the outcome of the study were the water treatment facility capital cost, raw / finished water piping capital cost, and plant staff operating cost. For the Motts Run WTP alternative, or the Abel Lake WTF or Tank sites, one of the reasons for the large cost differential relative to the Rocky Pen Run Reservoir options was the need to convey 21+ mgd of raw water to the treatment facility site, which involved long lengths of 36-inch diameter piping. Conveying raw water from Abel Lake to a treatment facility at Rocky Pen Run Reservoir also involved long pipelines, but the pipe diameter to convey 6 mgd is much smaller (assumed to be 16-inch). The capital cost associated with the water treatment facilities themselves also played a major role, because the unit cost to construct treatment facilities in Spotsylvania County was assumed to be much higher than the cost to construct similar sized treatment facilities in Stafford County.

Because operating costs were a significant portion of the total project cost of some alternatives (i.e., roughly one-third of the total cost), a sensitivity analysis was performed to assess the influence of the evaluation period and real discount rate on the present value of the operating costs. While the majority of the present value calculations were performed assuming a real discount rate of 3%, one present worth value calculation was made assuming a real discount rate of 2%. Under the latter scenario, while the total project cost of each alternative increased, the relative spread between costs was not changed (i.e., the Motts Run was still the most expensive alternative and the remaining alternatives were fairly close together in cost). Therefore, the selection of which real discount rate to use in the calculation of present value had no influence on the outcome of the siting study. Similarly, three sensitivity runs were made assuming the present value evaluation period was 20, 30, and 40 years, respectively. The results of the sensitivity analyses for the present value periods are summarized in Figure 1. The use of various present value periods generally did not appear to affect the selection of the preferred option. However, because present value operating costs made up roughly one-third of the total project cost for Alternative #1, the impact of the present value evaluation period was more significant for this alternative. As shown in Figure 1, this option would have been the preferred option for all periods up to 40 years.

Based on the costs and the results of the sensitivity analysis, it was decided at the May 22 progress review meeting that the Motts Run WTP alternative should be removed from further consideration. In addition, based on a detailed review of the siting analysis, the participants at the May 22 meeting concluded that Rocky Pen Run Reservoir is the preferred site for the new WTP. While the cost is very similar to other options, the Rocky Pen Run Reservoir site is preferred because it provides/facilitates:

- Enhanced reliability (3 plants vs. 2 plants).
- Enhanced security (on-site staffing at all 3 plants/reservoirs).
- Phased development of the transmission system.

- The ability to treat the raw water close to the demand centers.
- Ease of converting to a two plant operation in the future, by running a raw water pipeline from Abel Lake to the Rocky Pen Run Reservoir WTF, as noted below.

Initially, a water treatment facility with roughly 10 mgd of capacity could be constructed at the Rocky Pen Run Reservoir site to maximize use of the existing piping in the southern part of the County. It was suggested at the meeting that the initial phase include two process trains for reliability. Under the three plant alternative, the future Rocky Pen Run Reservoir WTF could serve the southern part of the County, the Abel Lake WTF could serve the areas north of Abel Lake, and the Smith Lake WTF could serve the northern part of the County. This approach minimizes the amount of large piping required to move treated water from Rocky Pen Run Reservoir.

It should be noted that the decision to operate three treatment facilities rather than two treatment facilities could be deferred to a later date (i.e., defer a portion of the total treatment capacity at the selected location rather than construct the entire consolidated facility in the near term). If Stafford County decides to operate as a two-plant system in the future, the added cost is minor (roughly $5 million on average) in comparison to the overall project cost. The existing Abel Lake WTF could then be decommissioned and the necessary length of raw water piping and treatment capacity constructed.

Appendix A
for
Draft Technical Memorandum No. 3
Facility Assessment for Abel Lake WTF

1.0. Introduction

This memorandum documents the facility assessments performed to identify the location, capacity, and general condition of the County's water and sewer facilities. The information gathered as part of these assessments will be used in other facets of the Master Plan project to assist in the decision-making process relative to abandonment, retention, or expansion of the existing facilities in the near-term and long-term.

2.0. Abel Lake Water Treatment Facility

2.1. Plant Overview/History

The Abel Lake Water Treatment Facility (WTF), located in the central portion of Stafford County, supplies up to 6 million gallons per day (mgd) of treated drinking water to residents in the southern portion of the County. The plant, located off of Moorewood Lane, can be accessed from Route 1 by way of Mountain View Road, Enon Road, and Hulls Chapel Road (the latter intersects with Moorewood Lane). Treatment at the Abel Lake WTF consists of raw water contact (oxidation), followed by flash mixing, vertical turbine flocculation, clarification, dual media filtration and disinfection. Primary disinfection is accomplished through the use of chlorine (sodium hypochlorite), while secondary (residual) disinfection is accomplished through the use of chloramines.

The original 2 mgd Abel Lake WTF was completed in July 1982 and included construction of the raw water intake and pumping facilities, flash mix and flocculation facilities, one circular clarifier, four dual media filters (only two were piped up), and finished water pumping facilities. In June 1988, another clarifier and the piping for the remaining two filters were constructed, bringing the total plant capacity to 4 mgd. In August of that year, a study was completed which resulted in the re-rating of the dual media filters (from 4 to 6 gpm/sf), bringing the total treatment capacity to 6 mgd. Since that time, various plant improvements have been constructed, including the alum storage building (1984), raw water contact tank and potassium permanganate feed facility (1990), raw and finished water pumping facility modifications (1992), and aqueous ammonia feed facilities (2003).

2.2. Assessment Findings

A meeting was held on May 20, 2003 to interview treatment facility operations staff and perform a site walkthrough to assess the condition of the various plant components. The following section, grouped by plant component, describes the findings of the site walkthrough, summarizes the key issues raised by the plant manager, and identifies any planned capital expenditures for each component.

- Raw Water Intake and Pumping. Based on visual inspection, the raw water intake structure looked to be in fairly good condition with some minor corrosion of the raw water pump access hatches. Plant staff indicated that the raw water pipes are still in acceptable condition, with no observed reduction in capacity. The raw water intake gates are reportedly operable. The exposed raw water lines appear to need some minor painting. Stafford County is planning to replace Raw Water Pumps No. 1 and No. 2 in year 2008, and Raw Water Pump No. 3 in 2010.

- Potassium Permanganate Feed Facilities/Raw Water Contact Chamber. Based on observations made during the plant walkthrough, the contact chamber and feed facilities appear to be in good condition; however, plant staff may replace the existing potassium permanganate feeder.

- Flash Mix Basin. During the plant walkthrough, some cracking was observed at the top of the flash mix basin wall (around the base of the handrail). Plant staff indicated that the flash mix motor was replaced within the last 10 years and is still in good condition. The County plans on replacing the mixer itself, and the associated control panel, in the year 2015.

- Flocculators. The flocculation basins themselves were observed to be in good condition during the plant walkthrough, with no apparent cracking of the concrete. Plant staff indicated that the existing flocculators needed to be refurbished or replaced. The County had planned on replacing this equipment in year 2002; no information is available to indicate if this has been rescheduled.

- Clarifiers. The clarifiers appeared to be in good condition at the time of the plant walkthrough. Plant staff indicated that the interior of the basins needs to be coated to protect against possible corrosion (due to the low coagulation pH required in order to meet the enhanced coagulation requirements of the Stage 1 DBP Rule). In fact, everything in contact with process water downstream of the clarifiers needs to be coated, including clarified water collection troughs, backwash troughs, etc. Plant staff also indicated that the circular sludge scrapers need to be replaced (especially the scraper in Basin A, which is apparently quite pitted), which is currently planned for year 2008.

- Filters. The four dual media (anthracite-sand) filters appear to be in relatively good condition. Some staining was observed on the interior walls of the filter boxes, possibly due to the use of potassium permanganate. Some staining of the roof was also observed, which is likely due to clogged roof leaders (as opposed to a roof leak, since plant staff indicated the roof was completely replaced 10 years ago). The filter bottoms are Leopold dual-parallel lateral clay tile underdrains and were not visibly inspected at the time of the walkthrough.

  Plant staff indicated that they are planning on switching out the existing BIF cylinder actuators with motor operators in year 2004, and are also looking to upgrade the filter control panels sometime in the near future. The County is planning to replace the media in Filters No. 1 and No. 2 in year 2003 and the media in Filters No. 3 and No. 4 in year 2010. The County is also planning to replace the valve motor operators on some of the filter effluent valves in year 2004.

- Finished Water Pumping. Finished Water Pump No. 2 was installed during the construction of the initial phase of the Abel Lake Treatment Facility, and plans were made to replace it in year 2002 (the changeout had not been performed at the time of the site visit). Plans are in place to replace Finished Water Pump No. 3 in year 2008 and Finished Water Pump No. 1, which was installed in year 1990, in year 2010 due to excessive noise.

- Chemical Systems. During the plant walkthrough, possible chemical corrosion damage was observed on and/or around some of the chemical metering pumps, and one of the volumetric chemical feeders appeared to be bandaged together with duct tape. The County is planning to replace the chemical solution and transfer pumps, as well as the volumetric feeders, by the year 2012 (replacement of this equipment has been phased as identified in the attached Capital

Expenditure Schedule for the Abel Lake WTF). The County is also planning to replace both the caustic and alum feed systems by year 2005 (the revisions are currently being designed in-house by County staff).

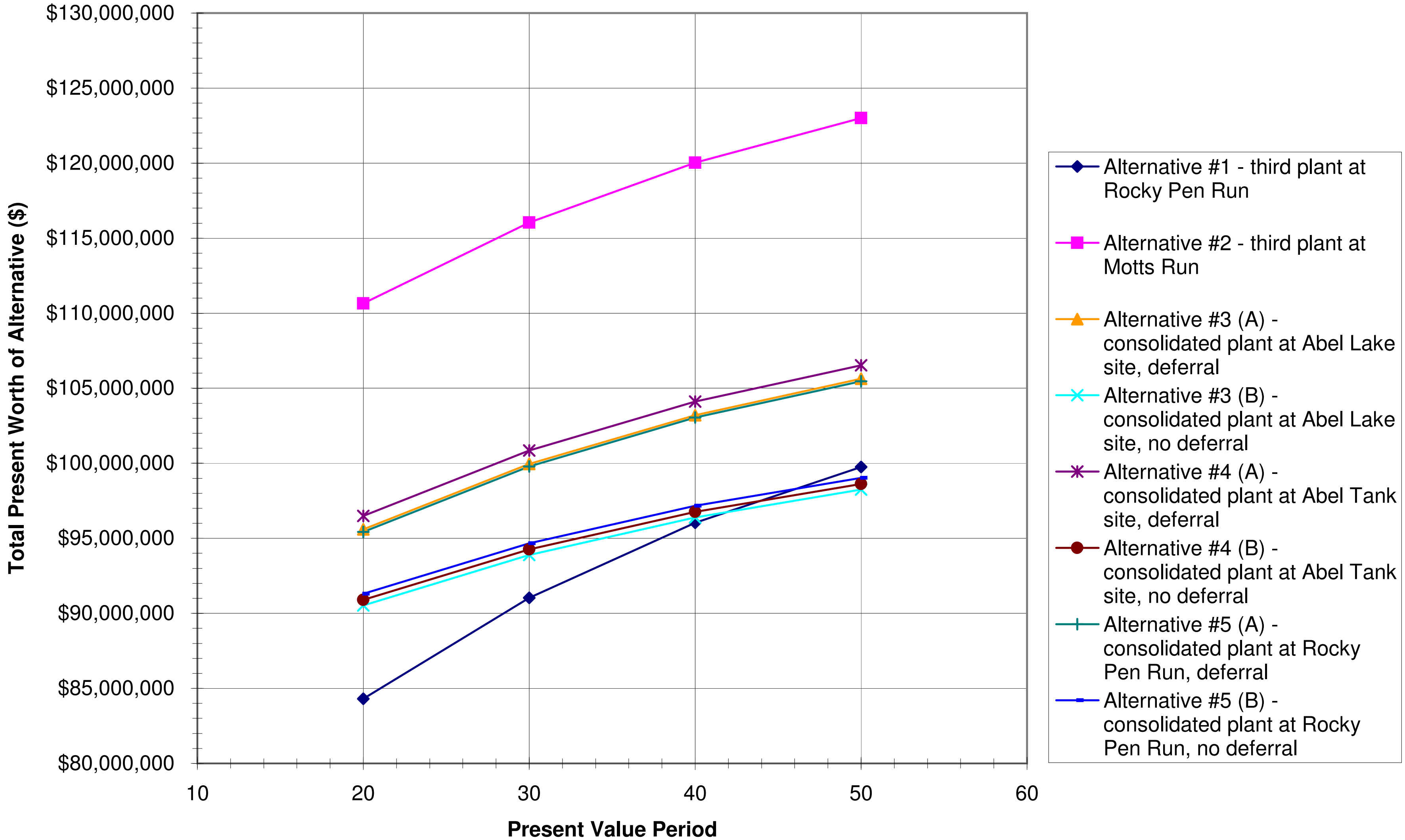
- Plant Power/Electrical System. Plant staff indicated that they have had problems with wire failures (e.g., Flocculator No. 2 has been prone to shorting out, often during rain events), which led operators to think that the failures may be due to corrosion of the underground conduits. The plant has budgeted for electrical service upgrades, including an emergency generator to power the facility, wiring improvements, and the cost of refurbishing the existing electrical room in order to bring it up to code. The County is also planning to replace the power safety failure valves at the facility in the year 2005.

- Miscellaneous. The County plans on repaving the plant entrance road in year 2005 and refurbishing the coal-tar roof in year 2006. Stafford County also plans on replacing the plant's booster pump system and two backflow preventers later this year. Finally, the County plans on replacing nine butterfly valves in year 2008. Other miscellaneous plant improvements have also been scheduled as identified in the attached capital expenditure schedule, supplied by Stafford County.

3.0. Conclusions

Overall, the facility appears to be in good condition and producing finished water of good quality. From the discussions at the meeting, it does not appear that the facility has any problems complying with current drinking water regulations. In general, based on visual inspection, the concrete structures appear to be in good condition. Much of the process equipment is original (installed circa 1982) and is nearing the end of its useful life; however, plans are in place to replace or refurbish this equipment in the next few years, as outlined in the attached capital expenditure schedule. If the planned improvements are made, along with periodic maintenance as required, it is expected that this facility could be maintained in service reliably through the planning period.

Figure 1. Effect of Present Value Period on Total Present Worth of Alternatives



Stafford County Water and Sewer Master Plan

Water Treatment Facility Siting Study - Site Comparison Matrix
20-year present value, 3% real discount rate

		Comparison Factors[2]						Markups							
		Capital				Present Worth O&M									
Alternative[3]	WTF Capacity (MGD)[1]	WTFs ($)	Pumping ($)	RW and FW Pipes ($)	Upfront Capital ($)	Staffing	Power	Subtotal ($)	Contingencies (20% of Subtotal) ($)	Total Capital Cost ($)	Engineering Legal & Admin (20% of Total Cap) ($)	Total Project Cost ($)	Unit Cost Based on Adj. Project Cost ($/gallon)	Present Value of Annual Operation & Maintenance ($)	Total Present Value of Alternative
Alternative 1: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Rocky Pen Run)	21.6	27,650,000	6,480,000	9,548,750	175,000	21,167,493	0	43,853,750	8,770,750	52,624,500	10,524,900	**63,149,400**	**$2.92**	**$21,167,493**	**$84,316,892.70**
Alternative 2: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Motts Run)	21.6	36,722,000	6,480,000	21,709,500	175,000	16,933,994	0	65,086,500	13,017,300	78,103,800	15,620,760	**93,724,560**	**$4.34**	**$16,933,994**	**$110,658,554.16**
Alternative 3: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Lake Site															
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	27.6	35,809,335	7,380,000	13,225,500	412,500	13,758,870	0	56,827,335	11,365,467	68,192,803	13,638,561	**81,831,363**	**$2.96**	**$13,758,870**	**$95,590,233.29**
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	34,500,000	7,380,000	13,225,500	412,500	10,583,746	0	55,518,000	11,103,600	66,621,600	13,324,320	**79,945,920**	**$2.90**	**$10,583,746**	**$90,529,666.35**
Alternative 4: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Tank Site															
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	27.6	37,376,541	8,380,000	11,434,500	258,897	13,758,870	0	57,449,938	11,489,988	68,939,925	13,787,985	**82,727,911**	**$3.00**	**$13,758,870**	**$96,486,780.82**
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	35,700,000	8,380,000	11,434,500	258,897	10,583,746	0	55,773,397	11,154,679	66,928,076	13,385,615	**80,313,692**	**$2.91**	**$10,583,746**	**$90,897,438.03**
Alternative 5: Utilize 2 WTFs to treat raw water (Rocky Pen Run and Smith Lake)															
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	27.6	35,159,335	8,380,000	13,000,000	175,000	13,758,870	0	56,714,335	11,342,867	68,057,203	13,611,441	**81,668,643**	**$2.96**	**$13,758,870**	**$95,427,513.29**
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	34,500,000	8,380,000	13,000,000	175,000	10,583,746	0	56,055,000	11,211,000	67,266,000	13,453,200	**80,719,200**	**$2.92**	**$10,583,746**	**$91,302,946.35**

Notes:
1. Water Treatment Facility (WTF) capacities are shown after new construction (or expansion) is complete.
2. For assumptions related to the comparison factors, refer to the attached sheets.
3. For the five alternatives identified above, raw water will be transported as follows:
   Alternative 1 - the Rocky Pen Run WTF will treat the Rocky Pen Run raw water
   Alternative 2 - the Motts Run WTP will treat the Rocky Pen Run raw water
   Alternative 3 - The Abel Lake WTF will treat both Abel Lake and Rocky Pen Run Reservoir raw water (i.e., the Rocky Pen Run WTF is not constructed) - Abel Lake Site
   Alternative 4 - The Abel Lake WTF will treat both Abel Lake and Rocky Pen Run Reservoir raw water (i.e., the Rocky Pen Run WTF is not constructed) - Abel Tank Site
   Alternative 5 - The Rocky Pen Run WTF will treat both Abel Lake and Rocky Pen Run Reservoir raw water (i.e., Abel Lake WTF is not used)

Stafford County Water and Sewer Master Plan

Water Treatment Facility Siting Study - Site Comparison Matrix
30 year present value, 3% real discount rate

		Comparison Factors[2]						Markups							
		Capital				Present Worth O&M									
Alternative[3]	WTF Capacity (MGD)[1]	WTFs ($)	Pumping ($)	RW and FW Pipes ($)	Upfront Capital ($)	Staffing	Power	Subtotal ($)	Contingencies (20% of Subtotal) ($)	Total Capital Cost ($)	Engineering Legal & Admin (20% of Total Cap) ($)	Total Project Cost ($)	Unit Cost Based on Adj. Project Cost ($/gallon)	Present Value of Annual Operation & Maintenance ($)	Total Present Value of Alternative
Alternative 1: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Rocky Pen Run)	21.6	27,650,000	6,480,000	9,548,750	175,000	27,887,273	0	43,853,750	8,770,750	52,624,500	10,524,900	**63,149,400**	**$2.92**	**$27,887,273**	**$91,036,672.75**
Alternative 2: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Motts Run)	21.6	36,722,000	6,480,000	21,709,500	175,000	22,309,818	0	65,086,500	13,017,300	78,103,800	15,620,760	**93,724,560**	**$4.34**	**$22,309,818**	**$116,034,378.20**
Alternative 3: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Lake Site															
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	27.6	35,809,335	7,380,000	13,225,500	412,500	18,126,727	0	56,827,335	11,365,467	68,192,803	13,638,561	**81,831,363**	**$2.96**	**$18,126,727**	**$99,958,090.32**
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	34,500,000	7,380,000	13,225,500	412,500	13,943,636	0	55,518,000	11,103,600	66,621,600	13,324,320	**79,945,920**	**$2.90**	**$13,943,636**	**$93,889,556.37**
Alternative 4: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Tank Site															
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	27.6	37,376,541	8,380,000	11,434,500	258,897	18,126,727	0	57,449,938	11,489,988	68,939,925	13,787,985	**82,727,911**	**$3.00**	**$18,126,727**	**$100,854,637.85**
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	35,700,000	8,380,000	11,434,500	258,897	13,943,636	0	55,773,397	11,154,679	66,928,076	13,385,615	**80,313,692**	**$2.91**	**$13,943,636**	**$94,257,328.05**
Alternative 5: Utilize 2 WTFs to treat raw water (Rocky Pen Run and Smith Lake)															
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	27.6	35,159,335	8,380,000	13,000,000	175,000	18,126,727	0	56,714,335	11,342,867	68,057,203	13,611,441	**81,668,643**	**$2.96**	**$18,126,727**	**$99,795,370.32**
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	34,500,000	8,380,000	13,000,000	175,000	13,943,636	0	56,055,000	11,211,000	67,266,000	13,453,200	**80,719,200**	**$2.92**	**$13,943,636**	**$94,662,836.37**

Notes:
1. Water Treatment Facility (WTF) capacities are shown after new construction (or expansion) is complete.
2. For assumptions related to the comparison factors, refer to the attached sheets.
3. For the five alternatives identified above, raw water will be transported as follows:
   Alternative 1 - the Rocky Pen Run WTF will treat the Rocky Pen Run raw water
   Alternative 2 - the Motts Run WTP will treat the Rocky Pen Run raw water
   Alternative 3 - The Abel Lake WTF will treat both Abel Lake and Rocky Pen Run Reservoir raw water (i.e., the Rocky Pen Run WTF is not constructed) - Abel Lake Site
   Alternative 4 - The Abel Lake WTF will treat both Abel Lake and Rocky Pen Run Reservoir raw water (i.e., the Rocky Pen Run WTF is not constructed) - Abel Tank Site
   Alternative 5 - The Rocky Pen Run WTF will treat both Abel Lake and Rocky Pen Run Reservoir raw water (i.e., Abel Lake WTF is not used)

Stafford County Water and Sewer Master Plan

Water Treatment Facility Siting Study - Site Comparison Matrix
40 year present value, 3% real discount rate

		Comparison Factors[2]						Markups							
		Capital				Present Worth O&M									
Alternative[3]	WTF Capacity (MGD)[1]	WTFs ($)	Pumping ($)	RW and FW Pipes ($)	Upfront Capital ($)	Staffing	Power	Subtotal ($)	Contingencies (20% of Subtotal) ($)	Total Capital Cost ($)	Engineering Legal & Admin (20% of Total Cap) ($)	Total Project Cost ($)	Unit Cost Based on Adj. Project Cost ($/gallon)	Present Value of Annual Operation & Maintenance ($)	Total Present Value of Alternative
Alternative 1: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Rocky Pen Run)	21.6	27,650,000	6,480,000	9,548,750	175,000	32,887,420	0	43,853,750	8,770,750	52,624,500	10,524,900	**63,149,400**	**$2.92**	**$32,887,420**	**$96,036,820.19**
Alternative 2: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Motts Run)	21.6	36,722,000	6,480,000	21,709,500	175,000	26,309,936	0	65,086,500	13,017,300	78,103,800	15,620,760	**93,724,560**	**$4.34**	**$26,309,936**	**$120,034,496.15**
Alternative 3: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Lake Site															
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	27.6	35,809,335	7,380,000	13,225,500	412,500	21,376,823	0	56,827,335	11,365,467	68,192,803	13,638,561	**81,831,363**	**$2.96**	**$21,376,823**	**$103,208,186.16**
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	34,500,000	7,380,000	13,225,500	412,500	16,443,710	0	55,518,000	11,103,600	66,621,600	13,324,320	**79,945,920**	**$2.90**	**$16,443,710**	**$96,389,630.09**
Alternative 4: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Tank Site															
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	27.6	37,376,541	8,380,000	11,434,500	258,897	21,376,823	0	57,449,938	11,489,988	68,939,925	13,787,985	**82,727,911**	**$3.00**	**$21,376,823**	**$104,104,733.69**
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	35,700,000	8,380,000	11,434,500	258,897	16,443,710	0	55,773,397	11,154,679	66,928,076	13,385,615	**80,313,692**	**$2.91**	**$16,443,710**	**$96,757,401.77**
Alternative 5: Utilize 2 WTFs to treat raw water (Rocky Pen Run and Smith Lake)															
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	27.6	35,159,335	8,380,000	13,000,000	175,000	21,376,823	0	56,714,335	11,342,867	68,057,203	13,611,441	**81,668,643**	**$2.96**	**$21,376,823**	**$103,045,466.16**
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	34,500,000	8,380,000	13,000,000	175,000	16,443,710	0	56,055,000	11,211,000	67,266,000	13,453,200	**80,719,200**	**$2.92**	**$16,443,710**	**$97,162,910.09**

Notes:
1. Water Treatment Facility (WTF) capacities are shown after new construction (or expansion) is complete.
2. For assumptions related to the comparison factors, refer to the attached sheets.
3. For the five alternatives identified above, raw water will be transported as follows:
   Alternative 1 - the Rocky Pen Run WTF will treat the Rocky Pen Run raw water
   Alternative 2 - the Motts Run WTP will treat the Rocky Pen Run raw water
   Alternative 3 - The Abel Lake WTF will treat both Abel Lake and Rocky Pen Run Reservoir raw water (i.e., the Rocky Pen Run WTF is not constructed) - Abel Lake Site
   Alternative 4 - The Abel Lake WTF will treat both Abel Lake and Rocky Pen Run Reservoir raw water (i.e., the Rocky Pen Run WTF is not constructed) - Abel Tank Site
   Alternative 5 - The Rocky Pen Run WTF will treat both Abel Lake and Rocky Pen Run Reservoir raw water (i.e., Abel Lake WTF is not used)

Stafford County Water and Sewer Master Plan

Water Treatment Facility Siting Study - Site Comparison Matrix
50-year present value, 2% real discount rate

		Comparison Factors[2]						Markups							
		Capital				Present Worth O&M									
Alternative[3]	**WTF Capacity (MGD)[1]**	**WTFs ($)**	**Pumping ($)**	**RW and FW Pipes ($)**	**Upfront Capital ($)**	**Staffing**	**Power**	**Subtotal ($)**	**Contingencies (20% of Subtotal) ($)**	**Total Capital Cost ($)**	**Engineering Legal & Admin (20% of Total Cap) ($)**	**Total Project Cost ($)**	**Unit Cost Based on Adj. Project Cost ($/gallon)**	**Present Value of Annual Operation & Maintenance ($)**	**Total Present Value of Alternative**
Alternative 1: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Rocky Pen Run)	21.6	33,000,000	7,680,000	9,548,750	175,000	44,709,129	0	50,403,750	10,080,750	60,484,500	12,096,900	**72,581,400**	**$3.36**	**$44,709,129**	**$117,290,529.38**
Alternative 2: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Motts Run)	21.6	43,752,000	7,680,000	21,709,500	175,000	29,060,934	0	73,316,500	14,663,300	87,979,800	17,595,960	**105,575,760**	**$4.89**	**$29,060,934**	**$134,636,694.10**
Alternative 3: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Lake Site															
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	27.6	39,224,588	8,580,000	13,225,500	412,500	29,060,934	0	61,442,588	12,288,518	73,731,106	14,746,221	**88,477,327**	**$3.21**	**$29,060,934**	**$117,538,261.11**
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	39,500,000	8,580,000	13,225,500	412,500	22,354,565	0	61,718,000	12,343,600	74,061,600	14,812,320	**88,873,920**	**$3.22**	**$22,354,565**	**$111,228,484.69**
Alternative 4: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Tank Site															
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	27.6	41,384,588	9,580,000	11,434,500	258,897	29,060,934	0	62,657,985	12,531,597	75,189,582	15,037,916	**90,227,499**	**$3.27**	**$29,060,934**	**$119,288,432.79**
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	40,700,000	9,580,000	11,434,500	258,897	22,354,565	0	61,973,397	12,394,679	74,368,076	14,873,615	**89,241,692**	**$3.23**	**$22,354,565**	**$111,596,256.37**
Alternative 5: Utilize 2 WTFs to treat raw water (Rocky Pen Run and Smith Lake)															
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	27.6	39,224,588	9,580,000	13,000,000	175,000	29,060,934	0	61,979,588	12,395,918	74,375,506	14,875,101	**89,250,607**	**$3.23**	**$29,060,934**	**$118,311,541.11**
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	39,500,000	9,580,000	13,000,000	175,000	22,354,565	0	62,255,000	12,451,000	74,706,000	14,941,200	**89,647,200**	**$3.25**	**$22,354,565**	**$112,001,764.69**

Notes:
1. Water Treatment Facility (WTF) capacities are shown after new construction (or expansion) is complete.
2. For assumptions related to the comparison factors, refer to the attached sheets.
3. For the five alternatives identified above, raw water will be transported as follows:
   Alternative 1 - the Rocky Pen Run WTF will treat the Rocky Pen Run raw water
   Alternative 2 - the Motts Run WTP will treat the Rocky Pen Run raw water
   Alternative 3 - The Abel Lake WTF will treat both Abel Lake and Rocky Pen Run Reservoir raw water (i.e., the Rocky Pen Run WTF is not constructed) - Abel Lake Site
   Alternative 4 - The Abel Lake WTF will treat both Abel Lake and Rocky Pen Run Reservoir raw water (i.e., the Rocky Pen Run WTF is not constructed) - Abel Tank Site
   Alternative 5 - The Rocky Pen Run WTF will treat both Abel Lake and Rocky Pen Run Reservoir raw water (i.e., Abel Lake WTF is not used)

Stafford County Water and Sewer Master Plan

Water Treatment Facility Siting Study - Site Comparison Matrix
50 year present value, 3% real discount rate

		Comparison Factors[2]						Markups							
		Capital				Present Worth O&M									
Alternative[3]	**WTF Capacity (MGD)[1]**	**WTFs ($)**	**Pumping ($)**	**RW and FW Pipes ($)**	**Upfront Capital ($)**	**Staffing**	**Power**	**Subtotal ($)**	**Contingencies (20% of Subtotal) ($)**	**Total Capital Cost ($)**	**Engineering Legal & Admin (20% of Total Cap) ($)**	**Total Project Cost ($)**	**Unit Cost Based on Adj. Project Cost ($/gallon)**	**Present Value of Annual Operation & Maintenance ($)**	**Total Present Value of Alternative**
Alternative 1: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Rocky Pen Run)	21.6	27,650,000	6,480,000	9,548,750	175,000	36,607,999	0	43,853,750	8,770,750	52,624,500	10,524,900	**63,149,400**	**$2.92**	**$36,607,999**	**$99,757,399.47**
Alternative 2: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Motts Run)	21.6	36,722,000	6,480,000	21,709,500	175,000	29,286,400	0	65,086,500	13,017,300	78,103,800	15,620,760	**93,724,560**	**$4.34**	**$29,286,400**	**$123,010,959.58**
Alternative 3: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Lake Site															
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	27.6	35,809,335	7,380,000	13,225,500	412,500	23,795,200	0	56,827,335	11,365,467	68,192,803	13,638,561	**81,831,363**	**$2.96**	**$23,795,200**	**$105,626,562.69**
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	34,500,000	7,380,000	13,225,500	412,500	18,304,000	0	55,518,000	11,103,600	66,621,600	13,324,320	**79,945,920**	**$2.90**	**$18,304,000**	**$98,249,919.74**
Alternative 4: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Tank Site															
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	27.6	37,376,541	8,380,000	11,434,500	258,897	23,795,200	0	57,449,938	11,489,988	68,939,925	13,787,985	**82,727,911**	**$3.00**	**$23,795,200**	**$106,523,110.22**
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	35,700,000	8,380,000	11,434,500	258,897	18,304,000	0	55,773,397	11,154,679	66,928,076	13,385,615	**80,313,692**	**$2.91**	**$18,304,000**	**$98,617,691.42**
Alternative 5: Utilize 2 WTFs to treat raw water (Rocky Pen Run and Smith Lake)															
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	27.6	35,159,335	8,380,000	13,000,000	175,000	23,795,200	0	56,714,335	11,342,867	68,057,203	13,611,441	**81,668,643**	**$2.96**	**$23,795,200**	**$105,463,842.69**
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	34,500,000	8,380,000	13,000,000	175,000	18,304,000	0	56,055,000	11,211,000	67,266,000	13,453,200	**80,719,200**	**$2.92**	**$18,304,000**	**$99,023,199.74**

Notes:
1. Water Treatment Facility (WTF) capacities are shown after new construction (or expansion) is complete.
2. For assumptions related to the comparison factors, refer to the attached sheets.
3. For the five alternatives identified above, raw water will be transported as follows:
   Alternative 1 - the Rocky Pen Run WTF will treat the Rocky Pen Run raw water
   Alternative 2 - the Motts Run WTP will treat the Rocky Pen Run raw water
   Alternative 3 - The Abel Lake WTF will treat both Abel Lake and Rocky Pen Run Reservoir raw water (i.e., the Rocky Pen Run WTF is not constructed) - Abel Lake Site
   Alternative 4 - The Abel Lake WTF will treat both Abel Lake and Rocky Pen Run Reservoir raw water (i.e., the Rocky Pen Run WTF is not constructed) - Abel Tank Site
   Alternative 5 - The Rocky Pen Run WTF will treat both Abel Lake and Rocky Pen Run Reservoir raw water (i.e., Abel Lake WTF is not used)

Water Treatment Plant Costs

Alternative	Water Treatment Plant Costs				
	Capacity (mgd)	Unit	Quantity	Unit Cost ($)	Cost ($)
Alternative 1: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Rocky Pen Run) [2,3]	21.6	ls	1	27,650,000	27,650,000
Alternative 2: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Motts Run) [4]	21.6	ls	1	36,722,000	36,722,000
Alternative 3: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Lake Site [5]					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	21.6	ls	1	36,274,588	36,274,588
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	ls	1	34,500,000	34,500,000
Alternative 4: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Tank Site [6]					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	20.5	ls	1	38,109,588	38,109,588
Option B - No additional staff added to operate expanded plant, no deferral.	26	ls	1	35,700,000	35,700,000
Alternative 5: Utilize 2 WTFs to treat raw water (Rocky Pen Run and Smith Lake) [5]					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	21.6	ls	1	35,624,588	35,624,588
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	ls	1	34,500,000	34,500,000

Notes:

1. It is assumed that the water quality is the same at each raw water source.
2. The size of the Rocky Pen Run WTP in Alternative 1 is based on the following equation: 1.5*total demand - 14 mgd at Smith Lake - 6 mgd at Abel Lake, where total demand = 27.7 mgd.
3. The cost of new water treatment capacity is based on a unit cost of $1.25/gal.
4. The cost of new water treatment capacity at Motts Run WTP (in Spotsylvania County) is based on a unit cost of $1.67/gal (using construction cost).
5. The total WTP cost for a consolidated Abel Lake/Rocky Pen Run WTP includes the following:
   For Option A:
   - the cost of annual plant maintenance (for options where the ex. Abel Lake WTF will be used through the 50-year planning period) is assumed to be $650,000.
   - the cost of replacing the ex. 6 mgd Abel Lake WTF in year 2023 (assuming a treatment cost of $0.65/gallon).
   - the cost of a new 25.6 MGD WTF (assuming the headworks costs $18.96 million and the 25.6 mgd of treatment capacity costs $0.65/gallon).
   (the reader should note that the future cost of 6 mgd treatment in year 2023 has been converted to a present value using a 20 year period and a real discount rate of 2%)
   For Option B:
   - the cost of a new 31.6 MGD WTF (based on a plant unit cost of $1.25/gallon).
6. The total WTP cost for a consolidated Abel Lake/Rocky Pen Run WTP at the Abel Lake Tank site includes the same components as Alternative 3, except that the treatment capacity has been derated by 5% assuming that the backwash treatment method (i.e., membranes) will allow the backwash water (assumed to be 5% of total plant flow) to be sent to distribution system.

Water Treatment Plant Costs

Alternative	Water Treatment Plant Costs Capacity (mgd)	Unit	Quantity	Unit Cost ($)	Cost ($)
Alternative 1: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Rocky Pen Run) [2,3]	21.6	ls	1	27,650,000	27,650,000
Alternative 2: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Motts Run) [4]	21.6	ls	1	36,722,000	36,722,000
Alternative 3: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Lake Site [5]					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	21.6	ls	1	35,809,335	35,809,335
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	ls	1	34,500,000	34,500,000
Alternative 4: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Tank Site [6]					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	20.5	ls	1	37,376,541	37,376,541
Option B - No additional staff added to operate expanded plant, no deferral.	26	ls	1	35,700,000	35,700,000
Alternative 5: Utilize 2 WTFs to treat raw water (Rocky Pen Run and Smith Lake) [5]					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	21.6	ls	1	35,159,335	35,159,335
Option B - No additional staff added to operate expanded plant, no deferral.	27.6	ls	1	34,500,000	34,500,000

Notes:
1. It is assumed that the water quality is the same at each raw water source.
2. The size of the Rocky Pen Run WTP in Alternative 1 is based on the following equation: 1.5*total demand - 14 mgd at Smith Lake - 6 mgd at Abel Lake, where total demand = 27.7 mgd.
3. The cost of new water treatment capacity is based on a unit cost of $1.25/gal.
4. The cost of new water treatment capacity at Motts Run WTP (in Spotsylvania County) is based on a unit cost of $1.67/gal (using construction cost).
5. The total WTP cost for a consolidated Abel Lake/Rocky Pen Run WTP includes the following:
   For Option A:
   - the cost of annual plant maintenance (for options where the ex. Abel Lake WTF will be used through the 50-year planning period) is assumed to be $650,000.
   - the cost of replacing the ex. 6 mgd Abel Lake WTF in year 2023 (assuming a treatment cost of $0.65/gallon).
   - the cost of a new 25.6 MGD WTF (assuming the headworks costs $18.96 million and the 25.6 mgd of treatment capacity costs $0.65/gallon).
   (the reader should note that the future cost of 6 mgd treatment in year 2023 has been converted to a present value using a 20 year period and a real discount rate of 3%)
   For Option B:
   - the cost of a new 31.6 MGD WTF (based on a plant unit cost of $1.25/gallon).
6. The total WTP cost for a consolidated Abel Lake/Rocky Pen Run WTP at the Abel Lake Tank site includes the same components as Alternative 3, except that the treatment capacity has been derated by 5% assuming that the backwash treatment method (i.e., membranes) will allow the backwash water (assumed to be 5% of total plant flow) to be sent to distribution system.

Pumping Station and Intake Costs

Alternative	Finished Water Pumping Station Costs Capacity (mgd)	Unit	Quantity	Unit Cost ($)	Cost ($)	Raw Water Pumping Station Costs Capacity (mgd)	Unit	Quantity	Unit Cost ($)	Cost ($)	Total Cost ($)
Alternative 1: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Rocky Pen Run)	21.6	gal	21,600,000	0.15	3,240,000	21.6	gal	21,600,000	0.15	3,240,000	6,480,000
Alternative 2: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Motts Run)	21.6	gal	21,600,000	0.15	3,240,000	21.6	gal	21,600,000	0.15	3,240,000	6,480,000
Alternative 3: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Lake Site	27.6	gal	27,600,000	0.15	4,140,000	21.6	gal	21,600,000	0.15	3,240,000	7,380,000
Alternative 4: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Tank Site	27.6	gal	27,600,000	0.15	4,140,000	21.6 / 6	ls	1	4,240,000.00	4,240,000	8,380,000
Alternative 5: Utilize 2 WTFs to treat raw water (Rocky Pen Run and Smith Lake)	27.6	gal	27,600,000	0.15	4,140,000	21.6 / 6	ls	1	4,240,000.00	4,240,000	8,380,000

Notes:
1. The costs for expanding existing pumping stations are based on the findings from the facility assessments performed as part of the Master Plan.
2. The costs for construction of new pumping stations (before markups) is based on the following table:

PS Size	**Installed Cost**	
< 1 mgd	$ 300,000	
2 mgd	$ 500,000	
5 mgd	$ 1,000,000	
> or = 10 mgd	$ 0.15	per gallon

Pipeline Costs

Alternative	Pipeline Costs Diameter (in)	Unit	Quantity	Unit Cost ($)	Cost ($)
Alternative 1: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Rocky Pen Run)	36	lf.	16,700	199	9,548,750
	18	lf.	11,250	110	
	20	lf.	6,750	124	
	30	lf.	15,750	173	
	Route 17 crossing	ea	1	200,000	
	Route 1 crossing	ea	1	51200	
	I-95 crossing	ea	1	1175000	
Alternative 2: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Motts Run) [2]	36	lf.	49,950	199	21,709,500
	18	lf.	11,250	110	
	20	lf.	6,750	124	
	30	lf.	15,750	173	
	Route 17 crossing	ea	1	200,000	
	Route 1 crossing	ea	1	51200	
	I-95 crossing	ea	1	1175000	
Alternative 3: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Lake Site	36	lf.	36,750	199	13,225,500
	18	lf.	9,000	110	
	20	lf.	2,250	124	
	30	lf.	5,850	173	
	30	lf.	15,750	140	
	Route 17 crossing	ea	1	200,000	
	Route 1 crossing	ea	1	51200	
	I-95 crossing	ea	1	1175000	
Alternative 4: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Tank Site	36	lf.	27,750	199	11,434,500
	18	lf.	9,000	110	
	20	lf.	2,250	124	
	30	lf.	5,850	173	
	30	lf.	15,750	140	
	Route 17 crossing	ea	1	200,000	
	Route 1 crossing	ea	1	51200	
	I-95 crossing	ea	1	1175000	
Alternative 5: Utilize 2 WTFs to treat raw water (Rocky Pen Run and Smith Lake)	36	lf.	16,700	199	13,000,000
	16	lf.	30,000	96	
	20	lf.	6,750	124	
	24	lf.	11,250	143	
	30	lf.	15,750	173	
	Route 17 crossing	ea	2	200,000	
	Route 1 crossing	ea	1	51200	
	I-95 crossing	ea	1	1175000	

Notes:
1. The costs for pipelines are based on Tables 2 and 3 from Technical Memorandum No. 2. These tables are in turn based upon the unit cost information contained on the Unit Costs tab. Tables 2 and 3 are reproduced on this spreadsheet to facilitate real-time manipulation of pipeline costs.
2. The pipeline cost for the Motts Run WTP alternative includes the cost of a 9' diameter tunnel beneath the Rappahannock River to hold a 36" raw water line and a 36" finished water line. This cost is based on a unit price of $3,500/lf, assuming the tunnel route is completely in solid rock, the use of a tunnel-boring-machine to dig the tunnel.

Upfront Capital Expenditures

Alternative	Upfront Capital Costs Land Acquisition ($)	Site Development ($)	Total Cost ($)
Alternative 1: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Rocky Pen Run)	175000		175,000
Alternative 2: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Motts Run)	175000		175,000
Alternative 3: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Lake Site [1]	412500		412,500
Alternative 4: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Tank Site [2]	258897		258,897
Alternative 5: Utilize 2 WTFs to treat raw water (Rocky Pen Run and Smith Lake)	175000		175,000

Notes:

1. It is assumed, based on information provided by Stafford County, that there is sufficient land at the Motts Run WTP site to construct a plant expansion.
2. Abel Lake land acquisition costs are based on the property value for the adjacent 13.757 ac. The reader should note that the above acreage assumption does not include land costs for residuals dewatering facilities or land costs for construction of residuals storage facilities.
3. Abel Lake Tank site land acquisition costs assume that the 47.811 ac adjacent to the tank are purchased in their entirety, at a land value of $5,415/acre

Relative Staffing Impacts
20 year present value, 3% real discount rate

Alternative	Annual Cost Impacts for Plant Staff: Operations Staff	Annual Wage ($)	Total Cost	Present Worth Factor	Present Worth
Alternative 1: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Rocky Pen Run)	20	$ 71,139.40	1,422,788	14.88	$ 21,167,493
Alternative 2: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Motts Run)	16	$ 71,139.40	1,138,230	14.88	$ 16,933,994
Alternative 3: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Lake Site					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	13	$ 71,139.40	924,812	14.88	$ 13,758,870
Option B - No additional staff added to operate expanded plant, no deferral.	10	$ 71,139.40	711,394	14.88	$ 10,583,746
Alternative 4: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Tank Site					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	13	$ 71,139.40	924,812	14.88	$ 13,758,870
Option B - No additional staff added to operate expanded plant, no deferral.	10	$ 71,139.40	711,394	14.88	$ 10,583,746
Alternative 5: Utilize 2 WTFs to treat raw water (Rocky Pen Run and Smith Lake)					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	13	$ 71,139.40	924,812	14.88	$ 13,758,870
Option B - No additional staff added to operate expanded plant, no deferral.	10	$ 71,139.40	711,394	14.88	$ 10,583,746

Notes:
1. Water treatment facility staffing is based on 10 staff per facility (1 plant manager, 1 mechanic, and 8 operators)
2. The average total salary cost for a treatment facility employee is approximately $71,139.40.
3. It is assumed that an expanded Motts Run WTP or a consolidated treatment facility (with partial treatment capacity deferral) would require 3 new staff to handle the increase in capacity (one assistant plant manager and two new mechanics/electricians).

Relative Staffing Impacts
30 year present value, 3% real discount rate

Alternative	Annual Cost Impacts for Plant Staff: Operations Staff	Annual Wage ($)	Total Cost	**Present Worth Factor**	**Present Worth**
Alternative 1: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Rocky Pen Run)	20	$ 71,139.40	1,422,788	19.60	$ 27,887,273
Alternative 2: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Motts Run)	16	$ 71,139.40	1,138,230	19.60	$ 22,309,818
Alternative 3: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Lake Site					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	13	$ 71,139.40	924,812	19.60	$ 18,126,727
Option B - No additional staff added to operate expanded plant, no deferral.	10	$ 71,139.40	711,394	19.60	$ 13,943,636
Alternative 4: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Tank Site					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	13	$ 71,139.40	924,812	19.60	$ 18,126,727
Option B - No additional staff added to operate expanded plant, no deferral.	10	$ 71,139.40	711,394	19.60	$ 13,943,636
Alternative 5: Utilize 2 WTFs to treat raw water (Rocky Pen Run and Smith Lake)					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	13	$ 71,139.40	924,812	19.60	$ 18,126,727
Option B - No additional staff added to operate expanded plant, no deferral.	10	$ 71,139.40	711,394	19.60	$ 13,943,636

Notes:
1. Water treatment facility staffing is based on 10 staff per facility (1 plant manager, 1 mechanic, and 8 operators)
2. The average total salary cost for a treatment facility employee is approximately $71,139.40.
3. It is assumed that an expanded Motts Run WTP or a consolidated treatment facility (with partial treatment capacity deferral) would require 3 new staff to handle the increase in capacity (one assistant plant manager and two new mechanics/electricians).

Relative Staffing Impacts
40 year present value, 3% real discount rate

Alternative	Annual Cost Impacts for Plant Staff Operations Staff	 Annual Wage ($)	 Total Cost	**Present Worth Factor**	**Present Worth**
Alternative 1: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Rocky Pen Run)	20	$ 71,139.40	1,422,788	23.11	$ 32,887,420
Alternative 2: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Motts Run)	16	$ 71,139.40	1,138,230	23.11	$ 26,309,936
Alternative 3: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Lake Site					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	13	$ 71,139.40	924,812	23.11	$ 21,376,823
Option B - No additional staff added to operate expanded plant, no deferral.	10	$ 71,139.40	711,394	23.11	$ 16,443,710
Alternative 4: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Tank Site					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	13	$ 71,139.40	924,812	23.11	$ 21,376,823
Option B - No additional staff added to operate expanded plant, no deferral.	10	$ 71,139.40	711,394	23.11	$ 16,443,710
Alternative 5: Utilize 2 WTFs to treat raw water (Rocky Pen Run and Smith Lake)					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	13	$ 71,139.40	924,812	23.11	$ 21,376,823
Option B - No additional staff added to operate expanded plant, no deferral.	10	$ 71,139.40	711,394	23.11	$ 16,443,710

Notes:
1. Water treatment facility staffing is based on 10 staff per facility (1 plant manager, 1 mechanic, and 8 operators)
2. The average total salary cost for a treatment facility employee is approximately $71,139.40.
3. It is assumed that an expanded Motts Run WTP or a consolidated treatment facility (with partial treatment capacity deferral) would require 3 new staff to handle the increase in capacity (one assistant plant manager and two new mechanics/electricians).

Relative Staffing Impacts
50-year present value, 2% real discount rate

Alternative	Annual Cost Impacts for Plant Staff: Operations Staff	Annual Wage ($)	Total Cost	Present Worth Factor	Present Worth
Alternative 1: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Rocky Pen Run)	20	$ 71,139.40	1,422,788	31.42	$ 44,709,129
Alternative 2: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Motts Run)	16	$ 71,139.40	1,138,230	31.42	$ 35,767,304
Alternative 3: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Lake Site					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	13	$ 71,139.40	924,812	31.42	$ 29,060,934
Option B - No additional staff added to operate expanded plant, no deferral.	10	$ 71,139.40	711,394	31.42	$ 22,354,565
Alternative 4: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Tank Site					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	13	$ 71,139.40	924,812	31.42	$ 29,060,934
Option B - No additional staff added to operate expanded plant, no deferral.	10	$ 71,139.40	711,394	31.42	$ 22,354,565
Alternative 5: Utilize 2 WTFs to treat raw water (Rocky Pen Run and Smith Lake)					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	13	$ 71,139.40	924,812	31.42	$ 29,060,934
Option B - No additional staff added to operate expanded plant, no deferral.	10	$ 71,139.40	711,394	31.42	$ 22,354,565

Notes:
1. Water treatment facility staffing is based on 10 staff per facility (1 plant manager, 1 mechanic, and 8 operators)
2. The average total salary cost for a treatment facility employee is approximately $71,139.40.
3. It is assumed that an expanded Motts Run WTP or a consolidated treatment facility (with partial treatment capacity deferral) would require 3 new staff to handle the increase in capacity (one assistant plant manager and two new mechanics/electricians).

Relative Staffing Impacts
50 year present value, 3% real discount rate

Alternative	Annual Cost Impacts for Plant Staff: Operations Staff	Annual Wage ($)	Total Cost	**Present Worth Factor**	**Present Worth**
Alternative 1: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Rocky Pen Run)	20	$ 71,139.40	1,422,788	25.73	$ 36,607,999
Alternative 2: Utilize 3 WTFs to treat raw water (Abel Lake, Smith Lake, and Motts Run)	16	$ 71,139.40	1,138,230	25.73	$ 29,286,400
Alternative 3: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Lake Site					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	13	$ 71,139.40	924,812	25.73	$ 23,795,200
Option B - No additional staff added to operate expanded plant, no deferral.	10	$ 71,139.40	711,394	25.73	$ 18,304,000
Alternative 4: Utilize 2 WTFs to treat raw water (Abel Lake and Smith Lake) - Abel Tank Site					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	13	$ 71,139.40	924,812	25.73	$ 23,795,200
Option B - No additional staff added to operate expanded plant, no deferral.	10	$ 71,139.40	711,394	25.73	$ 18,304,000
Alternative 5: Utilize 2 WTFs to treat raw water (Rocky Pen Run and Smith Lake)					
Option A - Additional staff to operate expanded plant, deferral of 6 mgd.	13	$ 71,139.40	924,812	25.73	$ 23,795,200
Option B - No additional staff added to operate expanded plant, no deferral.	10	$ 71,139.40	711,394	25.73	$ 18,304,000

Notes:
1. Water treatment facility staffing is based on 10 staff per facility (1 plant manager, 1 mechanic, and 8 operators)
2. The average total salary cost for a treatment facility employee is approximately $71,139.40.
3. It is assumed that an expanded Motts Run WTP or a consolidated treatment facility (with partial treatment capacity deferral) would require 3 new staff to handle the increase in capacity (one assistant plant manager and two new mechanics/electricians).

Abel Lake Water Treatment Facility

Assumed Inflation Rate: 3.0%

Capital Item	Location	Manufacturer	Model #	Year Installed	Original Cost	Estimated Useful Life, years	Estimated Replacement Year	Estimated Replacement Cost
Actuators		BIF (10)		1981	$25,000	22	2003	$47,903
Actuators		Limitorque	LY 2001 (4)	1994	$6,600	10	2004	$8,870
Alum Feed System		Raven	Ser. D22041	1985	$16,470	20	2005	$29,747
AWWA Butterfly Valves		Dezurik (9)		1988	$45,000	20	2008	$81,275
Backflow Preventers (2)		Watts	Series 900 RPZ 4"	1982	$1,000	20	2002	$1,806
Backwash Pumps		Worthington	SN-VTP 53806 #2	1981	$7,500	25	2006	$15,703
Backwash Pumps		Worthington	SN-VTP 53805 #1	1981	$7,500	25	2006	$15,703
Backwash System				1996	$7,400	15	2011	$11,529
Booster Pump System		Aurora	341A	1988	$2,500	15	2003	$3,895
Caustic Feed System		Plas-Tanks	PO# 387-01 Job 6987	1992	$12,000	10	2002	$16,127
Caustic Transfer Pump		Fybroc Division	1500 Ser# 921475	1992	$5,400	15	2007	$8,413
Chemical Room Hoist		Saturn Engineering	Serial # 79-9-4516	1982	$9,122	30	2012	$22,141
Chlorinators		Regal Toy 200	216	1988	$2,200	—		
Chlorinators		Superior	VR-56	2000	$875	—		
Chlorine Room Hoist		Saturn Engineering	Serial # 1646	1982	$3,416	—		
Clarifier		Envirex	NH46 Drive Half Bridge	1981	$75,000	20	2001	$135,458
Clarifier		Envirex	HYOHT 75'-0" Dia	1988	$80,835	20	2008	$145,997
Clearwell Baffles		Environetics	Hypalon ENV-3602-12	1994	$9,500	15	2009	$14,801
Dry Volumetric Feeders (5)		Wallace & Tiernan	Series 32-055	1982	$24,000	25	2007	$50,251
Entrance Road		P. C. Goodloe	Asphalt	1990	$17,116	15	2005	$26,666
Filter Effluent Valves		Limitorque	LY2001	1994	$1,650	10	2004	$2,217
Filters	Filters #1 & #2	F. B. Leopold	Turbikol 9500/Turbisand 4500	1981	$34,000	22	2003	$65,148
Filters	Filters #3 & #4	F. B. Leopold	Turbikol 9500/Turbisand 4500	1988	$34,000	22	2010	$65,148
Finished Pump #1		Worthington	S/N 92 tv-100791-1	1990	$40,000	20	2010	$72,244
Finished Pump #2		Worthington	S/N # 53803	1982	$7,752	20	2002	$14,001
Finished Pump #3		Worthington	S/N# 88TVU60410-1	1988	$32,000	20	2008	$57,796
Flash Mixer/Controller		Burhons-Sharp Co.	3N22-12	1995	$7,510	20	2015	$13,564
Flocculators		Envirex/Winsmith		1981	$21,471	21	2002	$39,942
Honeywell Recorder		Honeywell	TVMP-BO-80-A00-T00	2002	$7,500	10	2012	$10,079
Honeywell Recorder		Honeywell	TVMP-BO-80-A00-T00	2001	$6,650	10	2011	$8,937
Loss-of-Weight Recorder		Wallace & Tiernan	Series A-639	1982	$2,575	25	2007	$5,391
Power Safety Failure Valves				1995	$5,145	10	2005	$6,914
Raw Pumps		Hydromatic-SL8	RSBL 6000 M4-0	1988	$10,000	20	2008	$18,061
Raw Pumps (3)		Hydromatic-SL8	RSBL 6000 M4-0	1990	$10,000	20	2010	$18,061
Roof			Slag/Rubber Tar	1981	$25,000	25	2006	$52,344
Solution Pumps (2)		Wallace & Tiernan	Series 044-226 M9P	1982	$3,325	20	2002	$6,005
Solution Pumps (3)		Wallace & Tiernan	Series 044-12	1982	$4,820	30	2012	$11,699
Turbidimeters				2001	$5,400	10	2011	$7,257
Turbidimeters		Hach		2000	$10,380	10	2010	$13,950

Item	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015
Actuators			$47,903												
Actuators				$8,870											
Alum Feed System					$29,747										
AWWA Butterfly Valves								$81,275							
Backflow Preventers (2)		$1,806													
Backwash Pumps						$15,703									
Backwash Pumps						$15,703									
Backwash System											$11,529				
Booster Pump System			$3,895												
Caustic Feed System		$16,127													
Caustic Transfer Pump							$8,413								
Chemical Room Hoist												$22,141			
Chlorinators															
Chlorinators															
Chlorine Room Hoist															
Clarifier	$135,458														
Clarifier								$145,997							
Clearwell Baffles									$14,801						
Dry Volumetric Feeders (5)							$50,251								
Entrance Road					$26,666										
Filter Effluent Valves				$2,217											
Filters			$65,148												
Filters										$65,148					
Finished Pump #1										$72,244					
Finished Pump #2		$14,001													
Finished Pump #3								$57,796							
Flash Mixer/Controller															$13,564
Flocculators		$39,942													
Honeywell Recorder												$10,079			
Honeywell Recorder											$8,937				
Loss-of-Weight Recorder							$5,391								
Power Safety Failure Valves					$6,914										
Raw Pumps								$18,061							
Raw Pumps (3)										$18,061					
Roof						$52,344									
Solution Pumps (2)		$6,005													
Solution Pumps (3)												$11,699			
Turbidimeters											$7,257				
Turbidimeters										$13,950					

TECHNICAL MEMORANDUM 4

Development and Calibration of H2OMAP Water Hydraulic Model

Prepared for: Stafford County Department of Utilities
Prepare by: O'Brien & Gere
Date: November 2004

This technical memorandum is one of a series being prepared for the Stafford County Water and Sewer Master Plan project. The purpose of this technical memorandum is to summarize the development and calibration of the DOU's water system model. This technical memorandum discusses the data gathered as inputs into the model, summarizes the steps necessary to develop and verify the model data, and outlines the procedures followed to calibrate the model. At the conclusion of the steps described within this technical memorandum, a fully functional, calibrated model was established for DOU's water distribution and transmission system. The calibrated model will be used to analyze storage adequacy, low and high pressures, and fire flow adequacy. The model will be used to evaluate DOU's system in the current year (2003) and in the future (buildout) to identify the problem areas under a variety of demand conditions. Based on the model output, recommendations for minimizing the impacts of problem areas will be documented in Technical Memorandum 5 (*Finished Water Pumping, Storage and Distribution Facilities*).

This document contains the following sections:

Terminology, Definitions and Glossary

Average Dry Weather Flow (ADWF) – ADWF consists of average daily sewage flows and groundwater infiltration (GWI). ADWF is the average flow that occurs on a daily basis with no evident reaction to rainfall.

C-factor **–** A measure of the interior roughness of a pipe**.**

Diurnal Demand or Flow – Fluctuation of water demands or wastewater flows over a 24-hour period.

Effective Storage – Effective storage for each storage facility is determined by establishing the level in each tank above which all points in the water system can be served at 20 psi or higher (based on peak hour or maximum day plus fire flow).

Equalization Storage – The storage of peaking flows to prevent overflows from the sewer collection and conveyance systems.

Groundwater Infiltration (GWI) – Groundwater that infiltrates pipeline and manhole defects located below the ground surface. Groundwater infiltration is separate and distinguished from inflow resulting from storm events. Infiltration is a steady 24-hour flow that usually varies during the year in relation to the groundwater levels above the sewers. Infiltration rates are normally estimated from wastewater flows measured in the sewers during the early morning hours when water use is at a minimum and the flow is essentially infiltration.

H2OMAP Water – H2OMAP Water is a computer model used for modeling the Department of Utilities' water system under various demand conditions.

H2OMAP Sewer – H2OMAP Sewer is a computer model used for modeling the Department of Utilities' sewer system under various flow conditions.

Inflow – Drainage that enters the collection system through illegal or permitted connections, such as catch basins, downspouts, area drains and manhole covers. Inflow is separate and distinguished from infiltration. The inflow rate can be determined from the flow hydrographs recorded with flow meters by subtracting the normal dry weather flow and the infiltration from the measured flowrate.

Infiltration/Inflow (I/I) – The wastewater component caused by rainfall-dependent infiltration/inflow (RDI/I) and groundwater infiltration (GWI).

Maximum Day Demand **–** The one day in the year when the consumption is the highest.

Maximum Hour Demand **–** The one hour in the year when water consumption is the highest.

Node – A junction of two or more pipes, commonly representing a point where pipe characteristics change.

Peak Dry Weather Flow (PDWF) – PDWF consists of peak sewage flows plus GWI. PDWF is the highest measured hourly flow that occurs on a dry weather day.

Peak Wet Weather Flow (PWWF) – PWWF consists of ADWF plus RDI/I. PWWF is the highest measured hourly flow that occurs during wet weather.

Peak Factor – Peak factor is PWWF/ADWF.

Pressure Reducing Valve (PRV) – A valve that will maintain a specified downstream pressure.

Pressure Zone – A network of water pipes having a common static hydraulic grade line. Pressure zones are separated by closed valves, pressure regulating valves, pumping stations, and reservoirs.

Rainfall-Dependent Infiltration/Inflow (RDI/I) – RDI/I consists of rainfall that enters the collection system through direct connections (roof leaders, manholes, etc.) and causes an almost immediate increase in wastewater flow.

Service Area – The area served by the water distribution or wastewater collection system.

Steady State Simulation – A network model solution for a single point in time.

Tributary Area – The tributary area of a sewage system consists of all areas that contribute flow to the sewer by gravity and/or force main discharges.

ADD	Average Day Demand
ADWF	Average Dry Weather Flow
AWWA	American Water Works Association
CIP	Capital Improvement Program
cfs	Cubic Feet per Second
CMOM	Capacity, Management, Operation and Maintenance
CWA	Clean Water Act
DOU	Stafford County Department of Utilities
D/DBP	Disinfectants/Disinfection Byproducts
EA	Environmental Assessment
EIS	Environmental Impact Statement
EPA	US Environmental Protection Agency
EPS	Extended Period Simulation
ft	Feet
FY	Fiscal Year
gpcpd	Gallons per Capita per Day
gpd	Gallons per Day
gpm	Gallons per Minute
gpdidm	Gallons per Day per Inch Diameter – Mile
GWI	Groundwater Infiltration
HAAs	Haloacetic Acids
HGL	Hydraulic Grade Line
ICR	Information Collection Rule
I/I	Infiltration and Inflow
IESWTR	Interim Enhanced Surface Water Treatment Rule
ISO	Insurance Service Organization
L	Liter
MCL	Maximum Contaminant Level
MDD	Maximum Day Demand

MG	Million Gallons
MGD	Million Gallons Per Day
mg/l	Milligrams per Liter
mgd	Million Gallons per Day
MPN/100 ml	Most Probable Number per 100 Milliliters
NEPA	National Environmental Policy Act
O&M	Operations and Maintenance
PDWF	Peak Dry Weather Flow
PHD	Peak Hour Demand
PRV	Pressure Reducing Valve
psi	Pounds per Square Inch
PSV	Pressure Sustaining Valve
PWWF	Peak Wet Weather Flow
PWS	Public Water Supply
RDI/I	Rainfall-Dependent Infiltration/Inflow
SCADA	Supervisory Control and Data Acquisition
SDWA	Safe Drinking Water Act
SSO	Sanitary Sewer Overflows
SWTR	Surface Water Treatment Rule
TCR	Total Coliform Rule
THMs	Trihalomethanes
UFW	Unaccounted-for Water
ug/L	Micrograms per Liter
USACE	US Army Corps of Engineers
USEPA	US Environmental Protection Agency
USGS	US Geological Survey
VDEQ	Virginia Department of Environmental Quality
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant

Executive Summary

The overall objective for developing the hydraulic model of DOU's water distribution and transmission system is to assist in planning and prioritizing future capital improvements plan (CIP) projects within DOU's service area. The hydraulic model will be used to simulate flows within the transmission and distribution systems under existing (2003) and future (buildout) conditions. Based on the simulation output, recommendations will be developed that meet DOU's distribution system demands through buildout.

Development of the hydraulic water distribution model proceeded in three phases:

1. Data collection
2. Network development
3. Calibration

The data collection phase consisted of gathering DOU's best available data on their water distribution system to be modeled. The model network that was developed generally consisted of pipes and pumping stations. Specific data were gathered on the components of the network and incorporated into the H2OMAP Water model. Water connectivity was based on DOU's Geographic Information System (GIS) and system mapping. Water demands were input to the model using existing demand data from 2001 water billing data. Field tests were conducted for the water system to characterize the piping and included loss of head tests and hydrant flow tests. Fifteen C-factor and 20 hydrant flow tests were conducted to obtain the data needed to define the characteristics of the piping system and calibrate the model.

Roughly 80% of the water mains in the H2OMAP Water model were assigned C-factors of 130. Modeling runs conducted using alternative global C-factors indicated that the model is not particularly sensitive to moderate changes in pipe internal roughness (C-factors). In addition, field tests conducted for this Master Plan indicate that C-factors range from 130 to 150 and are expected to remain high through the planning period. Consequently, it was concluded that the overall C-factors for all of the pipes in the water model should be assigned a C-factor of 130 for planning purposes.

Modeling runs performed to calibrate the water model were based on pressures from the fire flow tests and the corresponding boundary conditions (i.e., pumping stations, water tanks, etc.). Modeled pressures at the fire flow test locations were found to be in good agreement with measured values (typically static pressure differences between the field tests and the model were within 4 psi and residual pressure differences were within 7 psi). Based on the modeling results, it was concluded that the H2OMAP Water model was adequately calibrated.

1.0. Data Collection

An uncalibrated hydraulic model of DOU's existing water distribution and transmission system was obtained from DOU at the outset of the study. For the existing water system, the pumping station, pipe, and storage tank information served as the physical foundation of the model. In addition to the physical data, field testing was performed to characterize the piping system and calibrate the model.

1.1. Pipe Network Data

The hydraulic model of DOU's water distribution and transmission system generally includes pipes 4 inches in diameter and larger. The key data for the pipes and nodes in the model include:

Pipes (links)
Pipe name
Upstream node
Downstream node
Cross section type
Pipe diameter
Pipe length

Junctions (nodes)
Node name
Ground surface elevation
X coordinate
Y coordinate

DOU provided these data which served as the physical foundation for the model.

1.2. Pumping Station Data

DOU currently operates nine water pumping stations located throughout the system.

- Smith Lake
- Moncure
- Vista Woods
- Abel Lake
- Berea
- Cranes Corner
- M&M
- Potomac Creek
- Mountain View

The H2OMAP Water model simulates the on/off operation of each individual pump, accounting for static and dynamic head and downstream losses. The data needed for physical pumps include pump on/off elevations and pump operating curves for each pump. The capacity and operating curves for the pumping stations were obtained from DOU.

1.3. Storage Tank Data

In addition to the pipelines and pumping stations, DOU currently operates 14 finished water storage facilities located throughout the system. Table 1 lists the key data for the storage tanks in the model.

Table 1: DOU distribution system storage facilities

System Component	Location	Maximum Water Level (feet)	Ground Elevation (feet)	Overflow Elevation (feet)	Volume (MG)
Stone River Tank	310 Zone	140	172	312	2.00
Courthouse Tank		60	250	310	0.25
Midway Tank		86.1	227	313.1	0.20
Moncure Tank		108.5	211.5	320	0.75
Vista Woods Tank	472 Zone	163.5	308.5	472	0.50
Shelton Shop Tank	433 Zone	95	338	433	1.375
Amyclae		155.31	282	437.31	1.50
Cranes Corner Tank	342 Zone	119	223.5	342.5	0.20
Bandy Tank		122.3	219	341.3	0.15
Ferry Road Tank		102.8	217.2	320	1.00
Grafton Tank		124	196.4	320.4	0.15
Berea Tank	503 Zone	150	353.5	503.5	0.50
Abel Lake Tank	342 Zone/503 Zone (pumped)	34	264	298	4.00
Smith Lake Tank	310 Zone	44	71	114.75	3.22

1.4. Water Demands

Calibration of the water model is based on existing water demands from water billing data (2001) which were provided by DOU. A detailed discussion of water demands is provided in Technical Memorandum 2 (*Water Demands*).

1.5. Field Testing

To obtain data to calibrate the H2OMAP Water model, field tests were conducted for the water system and included loss of head tests and hydrant flow tests. Fifteen C-factor and 20 hydrant flow tests were conducted to obtain the data needed to define the characteristics of the piping system.

2.0. Model Development

During model development, the inputs were established for the uncalibrated water model obtained from DOU.

2.1. Pipe Network Data and Connectivity

Pipes are conduits by which flow is transported by gravity or the energy supplied from pumps. DOU staff performed quality control checks on the pipe network data and connectivity during model construction. In addition, the H2OMAP software performs a number of quality control checks on the system during model applications, including checks on the connectivity of the system.

2.2. Water Demands

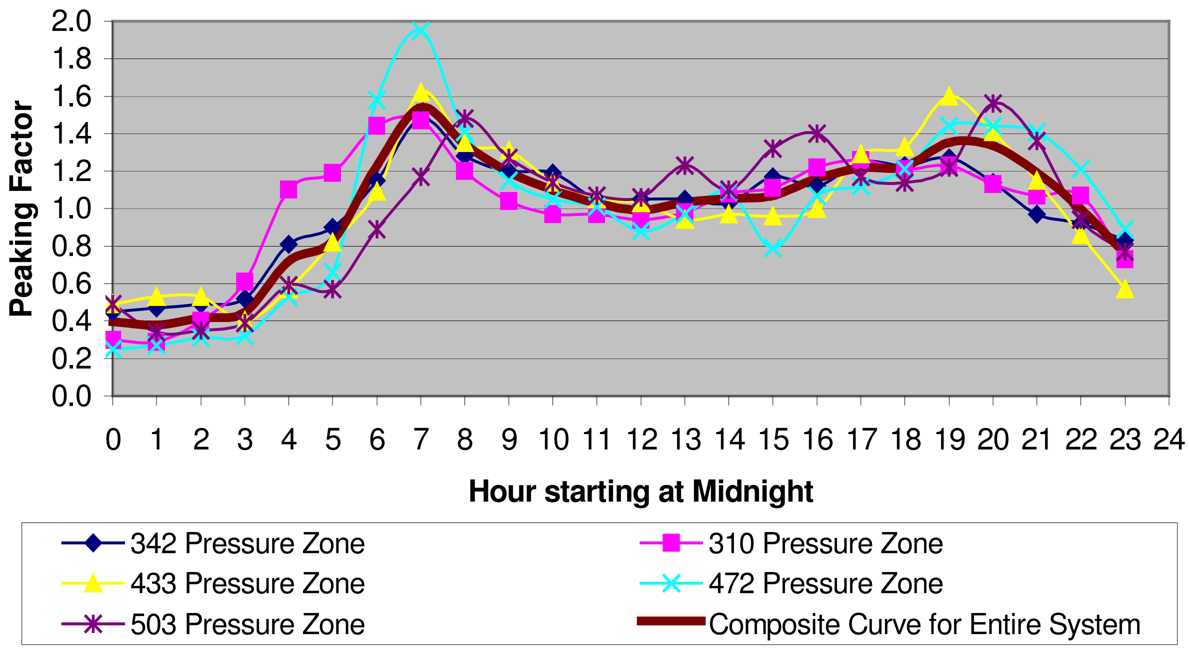
Water demands represent the average flows that are applied to the water system network from the contributing area. These demands are defined as the amount of water that must be carried by the distribution system to satisfy the need. Nodes represent points in the water system where water demands are taken from the system. For the model of the existing system which was used for calibration, DOU provided the water demands based on customer billing data for 2001. This approach results in an accurate allocation of water demands for model calibration.

For the period from July 2000 through December 2002, the total amount of water produced for the entire system (i.e., residential, commercial, industrial, etc.) was roughly 7.7 mgd which equates to roughly 122 gallons per day per person based on roughly 63,000 customers. When working with water records, water not accounted for needs to be considered. Unaccounted-for water is the difference between the total water supplied to the water systems from the water treatment plants and the amount of water measured from each of the individual water meters on each user's water connection. Although water distribution system leakage and meter inaccuracies are major reasons for water not being accounted for, there are many other causes. DOU estimates that roughly 17 percent of the water produced is currently unaccounted-for. For the period from July 2000 through December 2002, reducing the water produced by 17 percent to estimate customer water use (demands) results in a usage of 101 gallons per day per person based on 63,000 customers and an average water demand of 6.39 mgd (7.7 mgd x (100% - 17%)). This is a reasonable estimate of the per capita demand for the overall system (i.e., combined residential, industrial, commercial and institutional land uses).

2.3. Diurnal Demand Curves

Demands in water systems vary throughout the day with peaks in the morning and evening and low flows in the early morning hours. Patterns are used to represent the daily temporal variations within the water system. They consist of a collection of multipliers (multiplication factors) that are applied to the daily demand to allow it to vary over time during an extended period simulation (EPS). Different patterns can be applied to individual nodes or groups of nodes to accurately represent water duties (e.g., residential, commercial, etc.). For the calibration analysis, DOU developed diurnal demand curves for each of the five pressure zones based on monitoring data collected over a period of several days. The diurnal curves used in this Master Plan are shown in Figure 1. The diurnal curves used for modeling each pressure zone are based on combined demand categories (i.e., separate diurnal curves for various land use types such as residential and commercial were not generated). The diurnal curves were based on average hourly factors (pattern timestep in model) over a 24-hour period (duration in model). The diurnal demand curve was considered to be uniform throughout the pressure zone. Consequently, average daily water demands at nodes in each pressure zone were multiplied by their respective diurnal demand curve to generate daily variations in water demand.

Figure 1: Water System Diurnal Demand Curves



3.0. Field Testing

To obtain data to calibrate the H2OMAP Water model, the following field tests were conducted on April 7-10, 2003 for the piping in the water system:

- Loss of head tests
- Hydrant flow tests

Boundary conditions were monitored during the field testing program. The boundary condition points monitored included pumping and storage facilities from the SCADA system which provides continuous recording of data (20 minute data readings for facilities). This boundary condition information was used to calibrate the model against the data collected during the hydrant flow tests. The boundary conditions change with variations in the diurnal demand. The prevailing boundary information at the time each hydrant flow test was conducted was replicated in the H2OMAP Water model so that the test conditions were accurately simulated.

3.1. Loss of Head Tests (C-factor tests)

The C-factor of a pipe is a measure of pipe headloss caused by the internal roughness. Pipe roughness is a function of the age, material, and diameter of a pipe and typically increases with age. An increase in roughness correlates to a decrease in C-factor, which translates into increased headloss in the pipe. The requirements for a loss of head test include:

- Pipe diameter is known and must be the same throughout the length of pipe being tested. Pipes with larger diameter generally require higher flow rates and possibly longer lengths for measurable headloss. Pipe diameters for the field tests conducted in this Master Plan were obtained from DOU's GIS.

- Pipe length is known. Generally, pipe lengths are scaled from mapping. For this Master Plan, pipe length was verified by taking measurements in the field prior to testing.
- Pipe material should be the same throughout the pipe segment. Different materials will have different headloss characteristics. Typically, field testing is conducted on a representative sample of each pipe material type. However, DOU's water system pipe inventory in the GIS is incomplete (a significant portion of the piping is classified as "unknown" material type and the year installed is not known) so it was difficult to identify a representative sample of each pipe group. Rather than attempt to define a representative sample, field testing was conducted for at least one pipe segment within each group for mains with known size, material, and installation year.
- There should be few bends in the pipe segment so that the minor losses are not a measurable portion of the headloss.
- The quantity of flow remains constant along the pipe segment. To accomplish this, it is necessary to close valves so as to isolate the pipe segment. It is assumed that the individual demand from customers is insignificant during the testing.
- Velocity is under 10 ft/sec because the validity of the Hazen-Williams equation is inaccurate at higher velocities. In large pipes, a velocity of less than 2 ft/sec will yield insufficient headloss unless the segment of pipe is extremely long.
- The elevation at each end of the segment must be known precisely. Accuracy to within 0.1 ft might be needed. For this Master Plan, elevations at the ends of the pipe segment were obtained from DOU's GIS.
- Pressure at the endpoints of the pipe segment need to be measurable.

Because there are so many factors involved, it is difficult to obtain precisely all the data needed for the C-factor testing. Consequently, C-factor tests are sometimes conducted using erroneous or questionable data resulting in data that does not correlate well with other test data. In addition, headloss tests over a period of consecutive years may not produce a declining curve due to variability in the test results. Simply put, it is often difficult to conduct headloss tests that are accurate to within 5 points.

The Hazen-Williams C-factor can be calculated for a pipe segment as follows:

$$C_{H\text{-}W} = (42.73 \text{ x } Q) / d^{2.63} \text{ x } S^{.54})$$

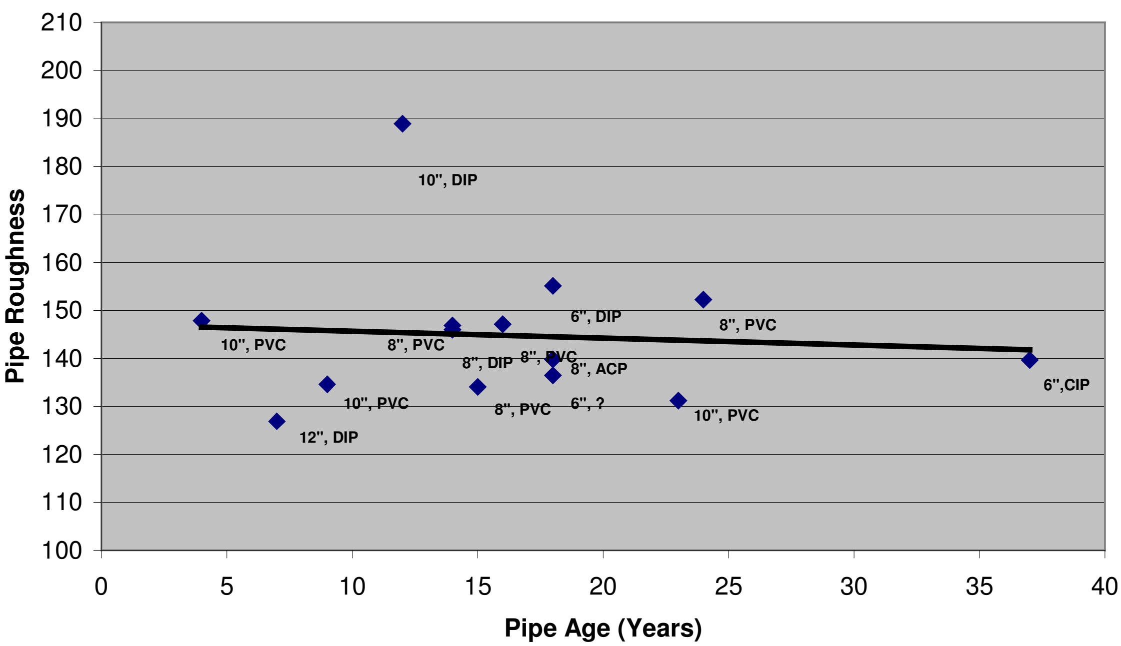
Where:
$C_{H\text{-}W}$ = Pipe roughness coefficient
Q = flow in pipe segment (gpm) measured by flowing a hydrant on the pipe segment
d = Diameter of pipe segment (inches)
S = Slope of hydraulic grade line (ft/100 ft)

A C-factor test can be conducted in the field by measuring the headloss over a segment of pipe. For this Master Plan, C-factor tests were performed using the parallel hose method. The parallel hose method consists of isolating a length of pipe between three fire hydrants. Flow is induced by opening one of the hydrants (called the flow hydrant), measuring the flowrate at the hydrant, and headloss between the hydrants. The headloss between the other two hydrants (called the pressure hydrants) is measured by running a hose from each hydrant to a U-Tube manometer (which is a direct measure of the headloss in the pipe). The Hazen-Williams equation is then solved using the measured headloss to determine the "C-factor" or roughness for the pipe. For each test, the flow and pressure hydrant and the valves to be closed to isolate the test section of pipe were identified by O'Brien & Gere. Tests were conducted in the field by O'Brien & Gere with assistance from DOU personnel. Each test was performed three times to confirm the accuracy of the "C-factor" measured.

A review of the input data for the H2OMAP Water model revealed that many of the mains (roughly 80%) were assigned C-factors of 130. A C-factor of this magnitude is generally reserved for relatively new or clean pipe. To verify the C-factors used in the H2OMAP Water model, loss of head tests were conducted in the field. To limit the number of tests, pipes of the same material type (cast, ductile, etc.), size, and age were grouped together, so that testing could be conducted on a limited sample of each group, and the results from the testing could then be applied to similar mains in the model. Using the grouping parameters, a total of 15 C-factor test locations were selected. The pipe diameters tested varied from 6-inch to 12-inch, and the age of pipes tested varied from 4 years to over 37 years.

Table 2 shows the results for the C-factor testing. At each test location, three tests were conducted at varying flowrates to verify the results. The C-factors for the pipes generally tested in a range between 125 and 155 (excludes the suspect C-factor test at Site No. 6). For the pipes tested, Figure 2 is a plot of the relationship of C-factor and age and shows a slight reduction in the C-factor values as the pipe age increases. The results of the field testing show that the C-factors range from 130 to 150 with little indication of a downward trend with age. DOU's water system contains a substantial amount of PVC pipe and DOU has identified that the ductile iron piping has a good layer of corrosion inhibitor on the inside of the pipe. Consequently, the C-factors for these two pipe categories are expected to remain high through the planning period. In addition, older cast-iron pipes with low C-factors will continue to be replaced and the amount of PVC piping with high C-factors is likely to increase. These actions will likely maintain C-factors for the overall system in the range produced during the field tests.

Figure 2: Hazen-Williams C-factor Test Results



3.2. Hydrant Flow Tests

Much of DOU's water system is composed of mains that are 12 inches or less in diameter. Therefore, fire flow conditions are an important hydraulic consideration in the evaluation of the system's capabilities. A total of 20 fire flow tests were conducted to collect the hydraulic information necessary to calibrate the model. The hydrant flow tests were conducted at the same time as the boundary conditions were being monitored, so that the hydraulic conditions in the system could be accurately simulated in the model for purposes of calibration.

Conducting fire flow tests is relatively simple. A pressure hydrant and a flow hydrant are selected. A pressure gage is installed on the pressure hydrant and a static pressure reading is taken. Once the static reading is taken, the flow hydrant can be opened to generate flow. A hand-held pitot gage is inserted in the flow stream and a flow reading is taken while the residual pressure is measured. If the differential pressure is less than 10 psi, consideration should be given to opening an additional flow hydrant to stress the system.

O'Brien & Gere identified the location of each fire flow test, including the locations of the flow hydrant and the residual hydrant. Testing was conducted in the field by O'Brien & Gere with assistance from DOU personnel. For each test, the static pressure was first recorded at the residual hydrant. The flow hydrant was opened until a steady flowrate was obtained at the flow hydrant and the residual pressure remained steady at the residual hydrant. The steady state flowrate and residual pressure were recorded. These fire flow field measurements were used to calibrate the model (Table 3).

Fire flow requirements are usually specified based on there being a residual pressure of 20 psi during an actual fire. The fire flow availability at 20 psi can be calculated at a hydrant as follows:

$$Q_{20} = Q_r \text{ x } (P_s\text{-}20)^{.54} / (P_s\text{-}P_r)^{.54}$$

Where:
Q_{20} = Available fire flow at 20 psi.
Q_r = Flow measured at the residual hydrant during field testing.
P_s = Pressure measured at the static hydrant during field testing.
P_r = Pressure measured at the residual hydrant during field testing.

The flow corresponding to a 20 psi residual pressure can be estimated at the test locations using the equation above or at each node in the system using the H2OMAP Water model.

4.0. Model Calibration

Calibration is the process of fine-tuning a model until it simulates field conditions for a specified time horizon (flow monitoring period) to an established degree of accuracy. Fine-tuning includes making minor adjustments to the input data to achieve the desired output data. The degree of accuracy refers to the difference between simulated and actual values. Generally, a model might be considered to be calibrated if simulated and measured pressures are within 5 to 10 psi.

One way to calibrate a water model is through a series of fire hydrant tests that measure flow and residual pressure at selected hydrants throughout the water system. For this Master Plan, a total of 20 fire flow tests were conducted. These tests were intended to stress the system and cause head losses in the vicinity of the test such that the impact of the pipe roughness coefficient is important.

As a first step in the model calibration process, steady state simulations were conducted under average day, maximum day and maximum hour conditions with the appropriate demand factors applied to the model to simulate the demand condition for the hour being modeled. Flows and pressures at the pumping stations and inflows and outflows at the tanks were compared with field data to confirm that the model was in general agreement with the field observations, and that the storage tanks were filling under low demand conditions and draining under the peak hour demands simulated.

Modeling runs were performed using the recorded fire flow results and the corresponding boundary conditions (see tank and pump tables at end of this technical memorandum). Modeled pressures at the fire flow test locations were found to be in good agreement with measured values (typically static pressure differences between the field tests and the model were within 4 psi and residual pressure differences were within 7 psi). Based on the modeling results shown in Table 4, it was concluded that the H2OMAP Water model was adequately calibrated for master planning.

Table 2
Hazen-Willimas C-factor Test Results

$C_{H\text{-}W} = (42.73 \times Q) / d^{2.63} \times S^{.54})$

Site No.	Location	Date	Pipe Size (in.)	Pipe Length (feet)	Deflection (" H2O)	Deflection (feet)	Slope of H.G. (ft./100 ft.)	Test Flow (gpm)	Calculated H-W Coefficient	Average H-W Coefficient	Year	Material	Comment
1	Richmond Dr.	4/7/2003	6.28	702	76.245	6.35	0.91	380	136.54				
	#2		6.28	702	149.175	12.43	1.77	530	132.54	136.47	1985	?	Operations personnel (inspectors) think this line was installed between 1969-79 & is ACP.
	#3		6.28	702	255.255	21.27	3.03	750	140.34				
2	Aquia Rd.	4/7/2003	8	480	20.995	1.75	0.36	450	139.80				
	#2		8	480	56.355	4.70	0.98	750	136.71	139.78	1985	ACP	
	#3		8	480	64.09	5.34	1.11	840	142.84				
3	Titanic Rd.	4/7/2003	9.86	754	13.26	1.11	0.15	380	111.43				
	#2		9.86	754	26.52	2.21	0.29	650	131.09	131.18	79-81	PVC	
	#3		9.86	754	45.305	3.78	0.50	1000	151.03				
5	Winding Creek Rd.	4/10/2003	12.58	1010	11.05	0.92	0.09	650	129.77				
	#2		12.58	1010	19.89	1.66	0.16	850	123.54	126.88	1996	DIP	
	#3		12.58	1010	25.415	2.12	0.21	1000	127.33				
6	Whitson Ridge Dr.	4/7/2003	10.52	651	33.15	2.76	0.42	1500	208.91				
	#2		10.52	651	56.355	4.70	0.72	1840	192.41	200.70	1991	DIP	
	#3		10.52	651	60.775	5.06	0.78	2000	200.79				
6	Whitson Ridge Dr.	4/10/2003	10.52	651	67.405	5.62	0.86	1900	180.38				Performed 2nd test due to "high" H-W result. Diameter may be incorrect or leaking valve.
	#2		10.52	651	82.875	6.91	1.06	2200	186.81	188.92	1991	DIP	
	#3		10.52	651	89.505	7.46	1.15	2450	199.57				
7	Sarasota Dr.	4/8/2003	8.51	759	20.995	1.75	0.23	380	128.51				
	#2		8.51	759	38.675	3.22	0.42	650	158.05	145.96	1989	DIP	
	#3		8.51	759	67.405	5.62	0.74	840	151.32				
9	Wining Colors Rd.	4/8/2003	8.04	1120	93.925	7.83	0.70	650	140.24				
	#2		8.04	1120	135.915	11.33	1.01	840	148.44	146.82	1989	PVC	
	#3		8.04	1120	180.115	15.01	1.34	1000	151.79				
12	Bryant Blyd.	4/8/2003	8.04	397	30.94	2.58	0.65	650	145.90				
	#2		8.04	397	51.935	4.33	1.09	840	142.54	147.12	1987	PVC	
	#3		8.04	397	62.985	5.25	1.32	1000	152.91				
13	Aurelie Dr.	4/8/2003	9.86	590	14.365	1.20	0.20	600	147.59				
	#2		9.86	590	29.835	2.49	0.42	840	139.25	147.85	1999	PVC	
	#3		9.86	590	46.41	3.87	0.66	1200	156.70				
14	Baldwin Dr.	4/9/2003	9.86	1320	44.2	3.68	0.28	650	134.61				
	#2		9.86	1320	16.575	1.38	0.10	380	133.65	134.59	1994	PVC	
	#3		9.86	1320	32.045	2.67	0.20	550	135.51				
15	Green Tree Rd.	4/9/2003	8.04	326	13.26	1.11	0.34	380	121.18				
	#2		8.04	326	28.73	2.39	0.73	650	136.53	134.08	1988	PVC	
	#3		8.04	326	45.305	3.78	1.16	880	144.53				
16	Lendall Ln.	4/9/2003	6.4	1070	68.51	5.71	0.53	380	172.81				
	#2		6.4	1070	250.835	20.90	1.95	650	146.67	155.14	1985	DIP	Inspectors think this line was constructed in 1976 & is ACP/DIP
	#3		6.4	1070	481.78	40.15	3.75	920	145.93				
17	Spring Valley Dr.	4/9/2003	6.4	700	46.41	3.87	0.55	380	169.59				
	#2		6.4	700	227.63	18.97	2.71	650	122.91	139.65	1966	CIP	
	#3		6.4	700	302.77	25.23	3.60	780	126.44				
18	Woodlawn Ter	4/10/2003	8.04	745	20.995	1.75	0.23	380	147.73				
	#2		8.04	745	48.62	4.05	0.54	650	160.57	152.25	1979	PVC	
	#3		8.04	745	124.865	10.41	1.40	1000	148.44				
19	Cool Spring Rd.	4/10/2003	12.58	977	24.31	2.03	0.21	1060	135.79	137.46		DIP	
	#2		12.58	977	44.2	3.68	0.38	1500	139.14				

- Denotes suspect values.

Table 3
Hydrant Flow Test Results

$Q_{20} = Q_r \times (P_s - 20)^{.54} / (P_s - P_r)^{.54}$

Site No.	Location	Date	Time	Test Static Pressure (psi)	Test Residual Pressure (psi)	Test Flow (gpm)	Flow at 20 psi Residual (gpm)	Tank Information	Pump Information
1	Richmond Dr.	4/7/2003	1000	60	52	750	1788.57	Moncure 23.7 Midway 31.9 StoneRiver 28.0	Moncure #2 on Smith Lake #2 on
2	Aquia Rd.	4/7/2003	1108	128	120	1000	4077.37	Moncure 23.7 Midway 31.7 StoneRiver 27.4	Moncure #2 on Smith Lake #2 on
3	Titanic Rd.	4/7/2003	1230	68	60	1000	2631.49	Moncure 23.7 Midway 32.0 StoneRiver 26.8	Moncure #2 on Smith Lake #2 on
4	Harpoon Dr.	4/7/2003	1400	112	100	750	2252.94	Moncure 23.7 Midway 31.8 StoneRiver 27.1	Moncure #2 on Smith Lake #2 on
5	Winding Creek Rd.	4/10/2003	1345	52	46	2330	5753.54		
6	Whitson Ridge Dr.	4/7/2003	1515	65	55	2000	4505.73	Shelton Shop 80.4	Moncure off Smith Lake #2 on
7	Sarasota Dr.	4/8/2003	834	68	60	2000	5262.98	Shelton Shop 82.7 AmyClae 35.0	Moncure #1 on Smith Lake #1 on
8	Larkwood Ct.	4/8/2003	911	80	68	2060	4912.60	Shelton Shop 83.0 AmyClae 34.8	Moncure #1 on Smith Lake #1 on
9	Wining Colors Rd.	4/8/2003	1048	76	64	1200	2757.05	Vista Woods 25.9	Moncure #1 on Smith Lake #1 on

Table 3
Hydrant Flow Test Results

$Q_{20} = Q_r \times (P_s\text{-}20)^{.54} / (P_s\text{-}P_r)^{.54}$

Site No.	Location	Date	Time	Test Static Pressure (psi)	Test Residual Pressure (psi)	Test Flow (gpm)	Flow at 20 psi Residual (gpm)	Tank Information	Pump Information
10	Aly Sheba Rd.	4/8/2003	1104	86	76	1250	3463.09	Vista Woods 26.7	Moncure #1 on Smith Lake #1 on
11	Vista Woods Rd.	4/8/2003	1138	80	74	1300	4507.58	Shelton Shop 84.3 Vista Woods 26.7	Moncure #1 on Smith Lake #1 on
12	Bryant Blyd.	4/8/2003	1126	71	60	2000	4578.95	Shelton Shop 85.6 Vista Woods 21.3	Moncure #1 on Smith Lake #1 on
13	Aurelie Dr.	4/8/2003	1433	79	58	1210	2113.72	Berea 28.8	Able Lake #1& #5 on
14	Baldwin Dr.	4/9/2003	1044	64	52	1060	2138.02	Berea 25.8	Able Lake #1 on
15	Green Tree Rd.	4/9/2003	1157	80	74	2700	9361.89	Berea 23.7	Able Lake #1& #5 on
16	Lendall Ln.	4/9/2003	1339	110	74	1130	1853.39	Bandy 19.6	Able Lake #1 on
17	Spring Valley Dr.	4/9/2003	1440	74	62	1190	2680.91	Cranes Corner 22.9	Able Lake #1 on
18	Woodlawn Ter	4/10/2003	857	64	54	1190	2648.57	Cranes Corner 20.2	Able Lake #1 on Ferry out of Serv.
19	Cool Spring Rd.	4/10/2003	955	112	90	1500	3248.09	Cranes Corner 19.6	Able Lake #1 on Ferry out of Serv.

Table 3
Hydrant Flow Test Results

$Q_{20} = Q_r \times (P_s-20)^{.54} / (P_s-P_r)^{.54}$

Site No.	Location	Date	Time	Test Static Pressure (psi)	Test Residual Pressure (psi)	Test Flow (gpm)	Flow at 20 psi Residual (gpm)	Tank Information	Pump Information
20	Colebrook Rd.	4/10/2003	1044	78	50	1000	1481.79	Cranes Corner 19.6	Able Lake #1 on Ferry out of Serv.

Table 4
Water Model Calibration Results

July 14, 2003

						Static Pressure/HGL							Residual Pressure/HGL							
						Pressure			Hydraulic Grade Line				Pressure				Hydraulic Grade Line			
Zone	Site No.	Static/Residual Hydrant ID	Static/Residual Node Number	Test Flow Hydrant ID	Flow Node Number	Test Static Pressure (psi)	Modeled Static Pressure (psi)	Static Pressure Difference (psi)	Elevation (ft)	Tested Static HGL (ft)	Modeled Static HGL (ft)	Static HGL Difference (ft)	Test Residual Pressure (psi)	Test Flow (gpm)	Modeled Residual Pressure (psi)	Residual Pressure Difference (psi)	Elevation (ft)	Tested Residual HGL (ft)	Modeled Residual HGL (ft)	Residual HGL Difference (ft)
310	2	7-H11-02	13000	7-H11-03	13382	128	124	4	16	312	302	9	120	1000	116	4	16	293	284	9
310	1	7-G12-09	13054	7-G11-04	13026	60	62	-2	**158**	297	301	-5	52	750	59	-7	158	278	294	-16
310	4	7-K12-01	13428	7-K12-02	13431	112	113	-1	43	302	304	-2	100	750	100	0	43	274	274	0
310	3	[1]	13522	7-K13-05	13520	68	71	-3	138	295	302	-7	60	1000	57	3	138	277	270	7
433	5	11-H1-01	20212	11-H1-02	21216	52	56	-4	301	421	430	-9	46	2330	47	-1	301	407	410	-2
433	8	7-B11-01	23136	7-A11-02	23126	80	83	-3	246	431	438	-7	68	2060	62	6	246	403	389	14
433	7	6-K12-07	25422	6-K12-05	25424	68	71	-3	270	427	434	-7	60	2000	62	-2	270	409	413	-5
433	6	[2]	25512	6-K11-11	25516	65	68	-3	260	410	416	-6	55	2000	53	2	260	387	383	4
472	11	6-E10-08	31014	6-D10-13	31012	80	80	0	281	466	466	0	74	1300	75	-1	281	452	454	-2
472	12	6-C11-11	31140	6-C10-05	31138	71	73	-2	292	456	461	-5	60	2000	64	-4	292	431	440	-9
472	9	10-K1-02	38014	5-K13-01	38030	76	75	1	288	464	461	2	64	1200	59	5	288	436	424	12
472	10	H38064A	38064	5-H12-02	38060	86	85	1	265	464	461	2	76	1250	71	5	265	441	429	12
342	19	19-C1-08	60602	16-J13-01	60600	112	112	0	67	326	326	0	90	1500	100	-10	67	275	297	-22
342	17	16-G9-03	61726	16-G9-04	61724	74	73	1	163	334	331	3	62	1190	64	-2	163	306	310	-3
342	18	[3]	63236	16-K11-09	63218	64	65	-1	176	324	325	-2	54	1190	56	-2	176	301	305	-4
342	20	[4]	65412	19-G3-05	65406	78	76	2	147	327	322	5	50	1000	50	0	147	263	262	1
342	16	16-D10-04	68002	16-D11-04	68002	110	109	1	81	335	332	3	74	1130	63	11	81	252	226	26
503	15	15-J4-21	73106	15-J5-01	73100	80	83	-3	300	485	492	-7	74	2700	74	0	300	471	471	0
503	14	15-H1-03	75018	15-H1-02	75012	64	66	-2	337	485	489	-5	52	1060	58	-6	337	457	471	-14
503	13	15-F3-04	76107	15-G3-01	76108	79	79	0	308	490	490	0	58	1210	63	-5	308	442	454	-12

Notes:
1 [1] = Intersection of Titanic Dr. & Raft Cove (appears to be Commodore Cove 7-K13-06)
2 [2] = Intersection of Whitson Ridge Dr. & Fieldstone Ct. (6-K11-09)
3 [3] = Intersection of Bridlepath Ln. & Woodlawn Ter. (16-K11-01)
4 [4] = Intersection of Ashbury Dr. & Briarwood Dr. (19-G3-11)

Water System Demand = 8.3 MGD

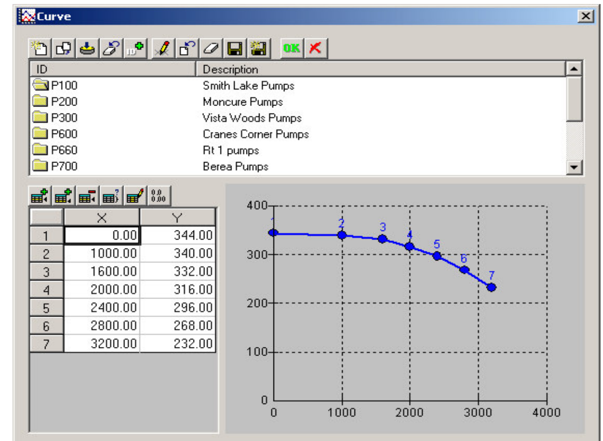
PUMP: ID (Char)	PUMP: DESCRIPT (Char)	PUMP: ZONE (Char)	OUTPUT: FLOW (gpm)	OUTPUT: HEADLOSS (ft)	PUMPHYD: CURVE (Char)
101	Smith Lake No. 1	310	2,842.83	264.15	P100
102	Smith Lake No. 2	310	2,842.83	264.15	P100
103	Smith Lake No. 3	310	0	0	P100
104	Smith Lake No. 4	310	0	0	P100
201	Moncure No. 1	433	1,418.65	165.17	P200
202	Moncure No. 2	433	1,418.65	165.17	P200
203	Moncure No. 3	433	0	0	P200
301	Vista Woods No. 1	472	608.16	40.79	P300
302	Vista Woods No. 2	472	0	0	P300
303	Vista Woods No. 3	472	0	0	P300
601	Cranes Corner No. 1	342	1,422.42	132.77	P600
602	Cranes Corner No. 2	342	1,422.42	132.77	P600
603	Cranes Corner No. 3	342	0	0	P600
701	Berea No. 1	503	1,629.48	250.53	P700
702	Berea No. 2	503	0	0	P700

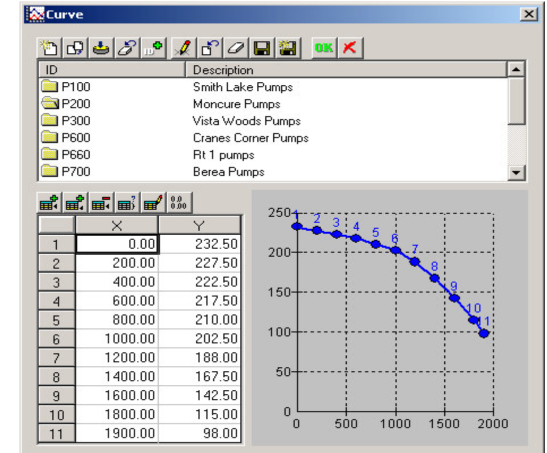
Notes:

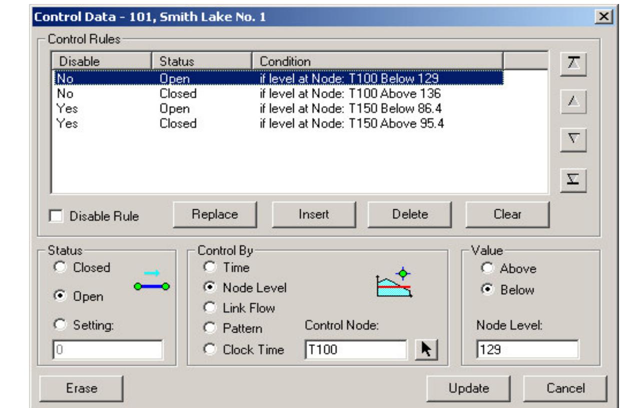
1. The following pumps are initially closed (turned off) in the model. They are turned on by their pump control settings.
   Pump 101 Smith Lake No. 1
   Pump 201 Moncure No. 1
   Pump 602 Cranes Corner No. 2

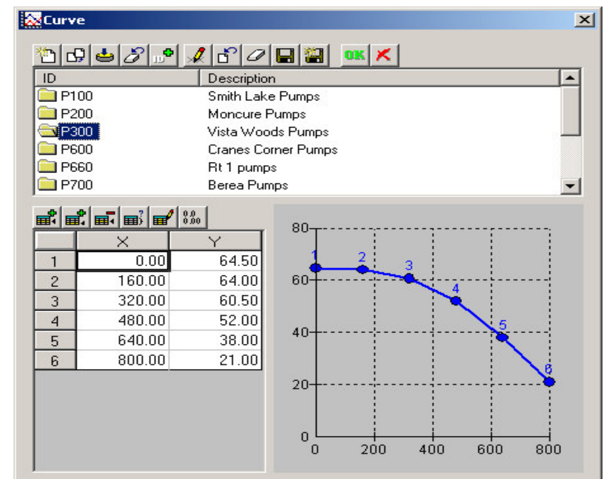
2. The following pumps are not inculded the present water system scenario (not assigned to a zone):
   661 M&M No.1
   680 Potomac Creek No. 1

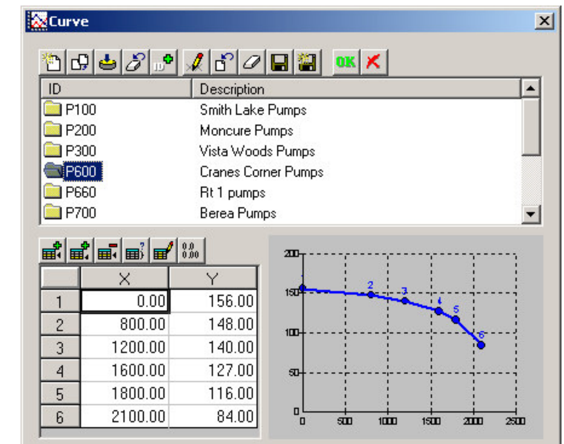
3. Pumps in Moncure Station were turned off in the model for Site 6.

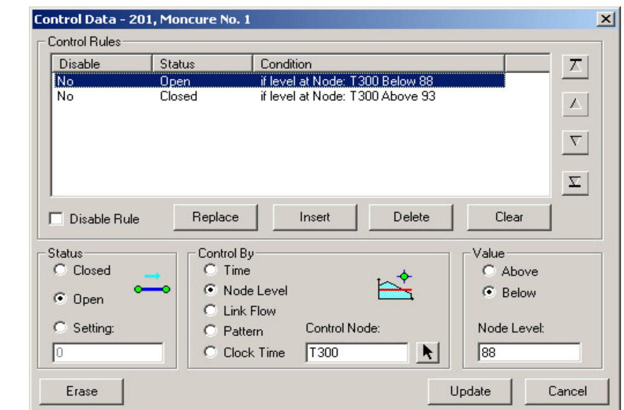


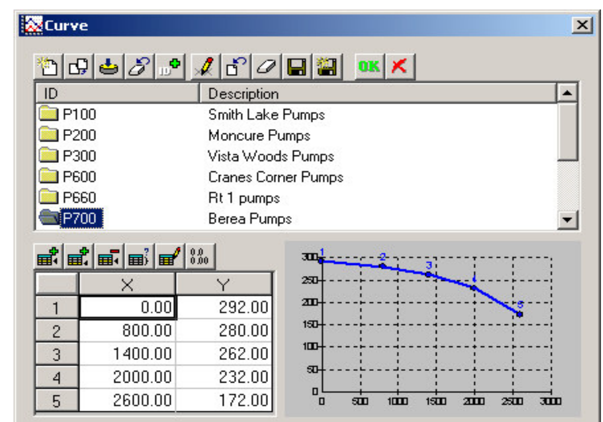


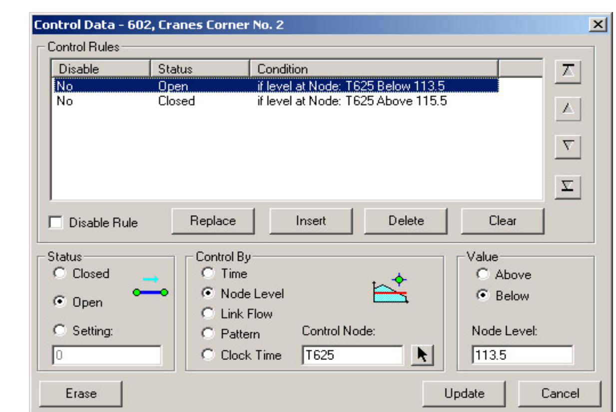












TANK: ID (Char)	TANK: DESCRIPT (Char)	TANK: ZONE (Char)	TANKHYD: ELEVATION (Num)	TANKHYD: MIN_LEVEL (Num)	TANKHYD: MAX_LEVEL (Num)	TANKHYD: INIT_LEVEL (Num)	TANKHYD: DIAMETER (Num)	TANKHYD: CURVE (Char)	OUTPUT: DEMAND (gpm)	OUTPUT: ELEVATION (ft)	OUTPUT: HEAD (ft)	OUTPUT: PRESSURE (psi)	OUTPUT: VOLUME (%)	OUTPUT: LEVEL (ft)
T100	Stone River Tank	310	172	100	140	128	97		318.65	172	300	55.46	70	128
T125	Courthouse Tank	310	250	0	60	55	26.6		-548.02	250	305	23.83	91.67	55
T150	Midway Tank	310	227	47.4	86.1	79.4	31.71		8.28	227	306.4	34.4	82.69	79.4
T180	Moncure Tank	433	211.5	68.5	108.5	92.2	64	T180	1,217.96	211.5	303.7	39.95	47.79	92.2
T200	Vista Woods Tank	472	308.5	125.9	163.5	151.9		T200	185.06	308.5	460.4	65.82	64.5	151.9
T300	Shelton Shop Tank	433	338	0	95	82	49.6		1,368.02	338	420	35.53	86.32	82
T60	Abel Lake	TO342503	264	0	0	12			-4,487.72	264	264	0	100	0
T625	Cranes Corner Tank	342	223.5	92.5	119	112.5	36		901.19	223.5	336	48.75	75.47	112.5
T650	Bandy Tank	342	219	93	122.3	113	30		345.62	219	332	48.96	68.26	113
T700	Ferry Road Tank	342	217.2	62.8	102.8	102.8		T700	-646.02	217.2	320	44.54	100	102.8
T725	Grafton Tank	342	196.4	96	124	124	32		-59.58	196.4	320.4	53.73	100	124
T900	Berea Tank	503	353.5	110	150	136		T900	891.51	353.5	489.5	58.93	64.84	136
T10	Smith Lake Tank	TO310	50	0	0	15			-5,685.66	50	50	0	100	0
T325	Amyclae	433	282	116	155.31	151	77	T325	-607.32	282	433	65.43	100	151
800		342	193.5	0	0	0	0		0	193.5	193.5	0	100	0
920	Celibrate Virginia	503	348	115	155	145	37.23		0	348	493	62.83	75	145
T190		310	200	70	110	100	50		0	200	300	43.33	75	100
T410	Embrey Mill	310	216	114	154	150	50		0	216	366	64.99	90	150

*Ferry Road Tank was taken off line in the model.

TECHNICAL MEMORANDUM 5

Finished Water Pumping, Storage and Distribution Facilities

Prepared for: Stafford County Department of Utilities
Prepare by: O'Brien & Gere
Date: November 2004

This technical memorandum is one of a series being prepared for the Stafford County Water and Sewer Master Plan project. The purpose of this technical memorandum is to summarize the performance of DOU's existing finished water pumping, storage and distribution facilities and to present recommendations for capital improvements to enhance system operations and performance, to accommodate for future growth and development, and to maintain system reliability and redundancy.

This document contains the following sections:

Terminology, Definitions and Glossary

Average Dry Weather Flow (ADWF) – ADWF consists of average daily sewage flows and groundwater infiltration (GWI). ADWF is the average flow that occurs on a daily basis with no evident reaction to rainfall.

C-factor – A measure of the interior roughness of a pipe.

Diurnal Demand or Flow – Fluctuation of water demands or wastewater flows over a 24-hour period.

Effective Storage – Effective storage for each storage facility is determined by establishing the level in each tank above which all points in the water system can be served at 20 psi or higher (based on peak hour or maximum day plus fire flow).

Equalization Storage – The storage of peaking flows to prevent overflows from the sewer collection and conveyance systems.

Groundwater Infiltration (GWI) – Groundwater that infiltrates pipeline and manhole defects located below the ground surface. Groundwater infiltration is separate and distinguished from inflow resulting from storm events. Infiltration is a steady 24-hour flow that usually varies during the year in relation to the groundwater levels above the sewers. Infiltration rates are normally estimated from wastewater flows measured in the sewers during the early morning hours when water use is at a minimum and the flow is essentially infiltration.

H2OMAP Water – H2OMAP Water is a computer model used for modeling the Department of Utilities' water system under various demand conditions.

H2OMAP Sewer – H2OMAP Sewer is a computer model used for modeling the Department of Utilities' sewer system under various flow conditions.

Inflow – Drainage that enters the collection system through illegal or permitted connections, such as, catch basins, downspouts, area drains and manhole covers. Inflow is separate and distinguished from infiltration. The inflow rate can be determined from the flow hydrographs recorded with flow meters by subtracting the normal dry weather flow and the infiltration from the measured flowrate.

Infiltration/Inflow (I/I) – The wastewater component caused by rainfall-dependent infiltration/inflow (RDI/I) and groundwater infiltration (GWI).

Maximum Day Demand – The one day in the year when the consumption is the highest.

Maximum Hour Demand – The one hour in the year when water consumption is the highest.

Node – A junction of two or more pipes, commonly representing a point where pipe characteristics change.

Peak Dry Weather Flow (PDWF) – PDWF consists of peak sewage flows plus GWI. PDWF is the highest measured hourly flow that occurs on a dry weather day.

Peak Wet Weather Flow (PWWF) – PWWF consists of ADWF plus RDI/I. PWWF is the highest measured hourly flow that occurs during wet weather.

Peak Factor – Peak factor is PWWF/ADWF.

Pressure Reducing Valve (PRV) – A valve that will maintain a specified downstream pressure.

Pressure Zone – A network of water pipes having a common static hydraulic grade line. Pressure zones are separated by closed valves, pressure regulating valves, pumping stations, and reservoirs.

Rainfall-Dependent Infiltration/Inflow (RDI/I) – RDI/I consists of rainfall that enters the collection system through direct connections (roof leaders, manholes, etc.) and causes an almost immediate increase in wastewater flow.

Service Area – The area served by the water distribution or wastewater collection system.

Steady State Simulation – A network model solution for a single point in time.

Tributary Area – The tributary area of a sewage system consists of all areas that contribute flow to the sewer by gravity and/or force main discharges.

ADD	Average Day Demand
ADWF	Average Dry Weather Flow
AWWA	American Water Works Association
CIP	Capital Improvement Program
cfs	Cubic Feet per Second
CMOM	Capacity, Management, Operation and Maintenance
CWA	Clean Water Act
DOU	Stafford County Department of Utilities
D/DBP	Disinfectants/Disinfection Byproducts
EA	Environmental Assessment
EIS	Environmental Impact Statement
EPA	US Environmental Protection Agency
EPS	Extended Period Simulation
ft	Feet
FY	Fiscal Year
gpcpd	Gallons per Capita per Day
gpd	Gallons per Day
gpm	Gallons per Minute
gpdidm	Gallons per Day per Inch Diameter – Mile
GWI	Groundwater Infiltration
HAAs	Haloacetic Acids
HGL	Hydraulic Grade Line
ICR	Information Collection Rule
I/I	Infiltration and Inflow
IESWTR	Interim Enhanced Surface Water Treatment Rule
ISO	Insurance Service Organization
L	Liter
MCL	Maximum Contaminant Level
MDD	Maximum Day Demand

MG	Million Gallons
MGD	Million Gallons Per Day
mg/l	Milligrams per Liter
mgd	Million Gallons per Day
MPN/100 ml	Most Probable Number per 100 Milliliters
NEPA	National Environmental Policy Act
O&M	Operations and Maintenance
PDWF	Peak Dry Weather Flow
PHD	Peak Hour Demand
PRV	Pressure Reducing Valve
psi	Pounds per Square Inch
PSV	Pressure Sustaining Valve
PWWF	Peak Wet Weather Flow
PWS	Public Water Supply
RDI/I	Rainfall-Dependent Infiltration/Inflow
SCADA	Supervisory Control and Data Acquisition
SDWA	Safe Drinking Water Act
SSO	Sanitary Sewer Overflows
SWTR	Surface Water Treatment Rule
TCR	Total Coliform Rule
THMs	Trihalomethanes
UFW	Unaccounted-for Water
ug/L	Micrograms per Liter
USACE	US Army Corps of Engineers
USEPA	US Environmental Protection Agency
USGS	US Geological Survey
VDEQ	Virginia Department of Environmental Quality
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant

Executive Summary

The Stafford County Department of Utilities' (DOU) water distribution system was evaluated as part of DOU's Water and Sewer Master Plan project. The findings and recommendations from this evaluation are summarized in this technical memorandum. Major topics include:

- Overview of DOU's existing finished water pumping, storage and distribution system and challenges facing the system.
- Performance of DOU's existing finished water pumping, storage and distribution facilities under near-term (2013) and buildout (2050) conditions and recommendations for capital projects to improve system operations and performance, to accommodate future growth and development, and to maintain system reliability and redundancy.

Overview of DOU's Water Distribution System

The DOU water supply system includes two water supply reservoirs (Abel Lake and Smith Lake), two water treatment plants, two large ground-level water storage tanks, six major water pumping stations, 12 elevated water storage tanks, and approximately 462 miles of pipes ranging in size from 4 to 30 inches in diameter. Most of the pipe material in the DOU distribution system is ductile iron pipe (DIP), cast iron pipe (CIP), asbestos-cement (A-C) pipe, and polyvinyl chloride (PVC) pipe.

DOU's current water distribution system is divided into five pressure zones essentially extending east and west from the Interstate 95 corridor:

- 310 Zone in the northeast portion of the County.
- 433 Zone in the northern portion of the County.
- 472 Zone in the northwest portion of the County.
- 342 Zone in the southeast portion of the County.
- 503 Zone in the southwest portion of the County.

Each of the water supply reservoirs has a water treatment facility adjacent to it. Although the water distribution system is interconnected, it is normally operated as two separate service areas. The total existing finished water storage in the distribution system is roughly 15.8 million gallons (mg).

Challenges of Future Service Area Growth and Development

The projected increase from the present water demand of approximately 8.4 mgd (2003) to 30.8 mgd over the next 47 years (through buildout in 2050) would represent an annual average increase of 0.47 mgd, which is consistent with the demand increase DOU has historically seen. Two critical conditions were considered: near-term which is just prior to Rocky Pen Run WTP coming online (2013) and buildout (2050). Near-term demands were based on 20 mgd maximum day water production capability (14 mgd from Smith Lake WTP and 6 mgd from Abel Lake WTP). Rocky Pen Run Reservoir, in combination with Smith Lake and Abel Lake, should be adequate to meet DOU's water needs through buildout (2050). Rocky Pen Run Reservoir is located in the southern portion of the County and will be primarily dedicated to the southern portion of the County under buildout conditions. Approximately 4 mgd of flow will need to be transferred from Rocky Pen Run WTP to the central portion of the service area that will be fed by the Abel Lake WTP. The flow balance for the near-term and buildout conditions is presented at the end of this technical memorandum in Appendix A.

Water systems are required to supply flow at rates that fluctuate over a wide range from day-to-day and hour-to-hour. Rates most important to planning, design and operation of a water system are average day, maximum (peak) day, maximum (peak) hour, and maximum hour plus fire flow.

- Average day demand is the total volume of water delivered to the system in a given year divided by the number of days in the year.
- Maximum (peak) day demand is the largest quantity of water supplied to the system on any given day of the year.
- Maximum (peak) hour demand is the highest rate of flow for any hour in a year.
- Maximum day plus fire flow considers the possibility of a fire event under maximum day demand conditions.

The peak day factor (maximum day demand / average day demand) for 2002 was 1.67. Peaking factors will drop as the system expands during the planning period. Average water demands are expected to increase from 8.4 mgd (2003) to roughly 30.8 mgd under buildout (2050) conditions. During the same period, the maximum day demands are expected to increase from approximately 13.0 mgd to 46.2 mgd at buildout (2050) based on a peaking factor of 1.5 times the average day demand. The current and projected demands are shown in Table 1.

Table 1: Current and projected water demands

Year	Average Day Water Demand (mgd) [1]	Maximum Day Water Demand (mgd) [2]
Current (2003)	8.4	13.0
Future (buildout at 2050)	30.8	46.2

[1] Maximum day and average day demands were determined based on calendar year.
[2] Maximum day finished water demands are based on representative historical one-day peaking factor of 1.5 developed based on historical DOU water production data.

In order to quantify the deficiencies in the existing water system and identify areas needing improvements under future conditions, the water distribution system was modeled using the MWH Soft's H2OMAP Water software. The hydraulic model was used to estimate the system's response to maximum day and peak hour demands, fire flows, and tank drawdown and refilling operations.

Maintaining Adequate Water Pumping, Storage and Distribution Facilities

With continued service area development and system expansion, additional storage will be required. DOU's existing 15.8 MG of storage volume satisfies the Virginia Department of Health's storage requirement to provide one-half of the average day demand (i.e., 4.2 MG in 2003). Finished water improvements are recommended to provide DOU with 21.5 MG of storage at buildout, which exceeds the VDH minimum storage requirement of 15.4 mg at buildout (i.e., one-half of anticipated average day demand of 30.8 mgd).

The piping in DOU's water distribution network must accommodate multiple objectives:

- Capacity – achieve adequate delivery capacity and acceptable pipeline head losses.
- Fire flow – supply fire flow at recommended levels.
- Growth – accommodate future service area development through system expansion.
- Looping – maintain water quality by looping and minimizing the number of "dead-ends" in the water distribution system.
- Redundancy – provide multiple points for delivery of water to areas.
- Reliability – maintain physical condition of system through proactive rehabilitation and replacement to minimize unscheduled loss of service.

Evaluations of DOU's water distribution system using the H2OMAP Water model indicate that while significant portions of the system meet each of these objectives reliably, improvements in other parts of the system are necessary to ensure that each of these objectives are met for current and future conditions.

Recommended Distribution System Capacity Improvements

This technical memorandum identifies the major components of DOU's water distribution system and evaluates the performance and operation of the system compared with the criteria presented in Section 5 (Review of Water Planning and Design Criteria) of this technical memorandum. The evaluation is based on a review of existing operational data, discussions with DOU staff, and results of the simulations from the H2OMAP Water modeling. The water system improvements presented in this technical memorandum are described in detail in Section 6 (Recommended Water System Improvements) and are shown on the figure in the pocket at the end of this Master Plan (Proposed Water System Improvements). The timing for implementation of the improvements is included in the pocket at the end of this Master Plan (Summary of Costs and Schedule for Recommended CIP Improvements (inside Urban Service Area)). The overall cost for the water system improvements needed to meet the projected growth through the buildout condition is roughly $51.4 million.

The water system improvements presented in this Master Plan are for the area inside the Urban Service Area. The piping needed to meet the water demands for the area outside the Urban Service Area were evaluated in order to properly size the transmission system needed to deliver flows to the outer areas of the County. Constructing large diameter pipes (12-inch and above) in the rural areas of the County to meet fire flows and other water system criteria could result in water quality degradation in the piping network in the rural areas due to long residence times caused by long lengths of large diameter pipe with low water demands. Consequently, careful planning will be needed if the water system is expanded to these rural areas outside the Urban Service Area.

1.0. Overview of Existing System

DOU's current water distribution system is divided into five pressure zones essentially extending east and west from the Interstate 95 corridor:

- 310 Zone in the northeast portion of the County.
- 433 Zone in the northern portion of the County.
- 472 Zone in the northwest portion of the County.
- 342 Zone in the southeast portion of the County.
- 503 Zone in the southwest portion of the County.

DOU has two raw water supply reservoirs: Smith Lake and Abel Lake in the northern and central portions of the County, respectively. Each of the water supply reservoirs has a water treatment facility adjacent to it. Although the water distribution system is interconnected, it is currently operated as essentially two separate service areas.

In general, finished water is pumped from the clearwells of the Smith Lake and Abel Lake WTP's to the distribution system as follows:

- Smith Lake WTP supplies three zones with hydraulic grade lines of 310, 433 and 472 feet. Water from Smith Lake WTP clearwells is pumped to a 3.3 MG ground level storage tank. Water from the 3.3. MG water tank near Smith Lake WTP is pumped to the 310 Zone and boosted from the 310 Zone to the 433 Zone through the Moncure Pumping Station. Water from the 433 Zone is boosted to the 472 Zone through the Vista Woods Pumping Station.

- Abel Lake WTP supplies water to two zones with hydraulic grade lines of 342 feet and 503 feet. Water from the Abel Lake WTP and pumping station is pumped through a 16-inch water main to the Abel Lake Tank. The Abel Lake Tank is a 4 MG ground level tank with an overflow elevation of 298 feet. Water is pumped from the Abel Lake Tank to the 342 Zone through the Cranes Corner Pumping Station and to the 503 Zone through the Berea Pumping Station.

1.1. Transmission and Distribution Mains

The hydraulic model of DOU's water distribution and transmission system generally includes pipes 4 inches in diameter and larger. The key data for the pipes and nodes in the model include:

Pipes (links)
Pipe name
Upstream node
Downstream node
Cross section type
Pipe diameter
Pipe length

Junctions (nodes)
Node name
Ground surface elevation
X coordinate
Y coordinate

DOU provided these data which served as the physical foundation for the model. The distribution of water mains included in the H2OMAP Water model are shown in Table 2.

Table 2: Distribution of water mains in the H2OMAP Water model

Pipe Size (inches)	Length (feet)	Percentage of Total Footage in Model
2	6,389	0.3%
4	37,522	1.9%
6	100,726	5.0%
8	1,537,903	76.0%
10	94,170	4.7%
12	54,304	2.7%
14	6,705	0.3%
15	31,874	1.6%
16	26,567	1.3%
18	65,202	3.2%
20	520	0.0%
21	6,840	0.3%
22	650	0.0%
24	44,205	2.2%
28	1383.2	0.1%
30	7,998	0.4%
Total	2,022,959	100.0%

1.2. Pumping Station Data

DOU currently operates six major water pumping stations located throughout the system which are shown in Table 3. Several pumping stations (Potomac Creek, M&M, and Mountain View Pumping Stations) are typically used for backup service.

Table 3: Water pumping stations

Pumping station	Capacity (mgd)
Smith Lake	10
Moncure	5.0
Vista Woods	1.0
Abel Lake	6.0
Berea	3.2
Cranes Corner	5.2

The H2OMAP Water model simulates the on/off operation of each individual pump, accounting for static and dynamic head and downstream losses. The data needed for physical pumps include pump on/off elevations and pump operating curves for each pump. The capacity and operating curves for the existing pumping stations were obtained from DOU and are shown in Appendix B of this technical memorandum.

1.3. Finished Water Storage

There are currently 14 finished water storage facilities in the DOU distribution system. The characteristics of each tank are summarized in Table 4.

Table 4: DOU distribution system storage facilities

System Component	Location	Maximum Water Level (feet)	Ground Elevation (feet)	Overflow Elevation (feet)	Volume (MG)
Stone River Tank	310 Zone	140	172	312	2.00
Courthouse Tank		60	250	310	0.25
Midway Tank		86.1	227	313.1	0.20
Moncure Tank		108.5	211.5	320	0.75
Vista Woods Tank	472 Zone	163.5	308.5	472	0.50
Shelton Shop Tank	433 Zone	95	338	433	1.375
Amyclae		155.31	282	437.31	1.50
Cranes Corner Tank	342 Zone	119	223.5	342.5	0.20
Bandy Tank		122.3	219	341.3	0.15
Ferry Road Tank		102.8	217.2	320	1.00
Grafton Tank		124	196.4	320.4	0.15
Berea Tank	503 Zone	150	353.5	503.5	0.50
Abel Lake	342 Zone/503 Zone (pumped)	34	264	298	4.00
Smith Lake	310 Zone	44	71	114.75	3.22

2.0. Level of Service Requirements

The level of service refers to the adequacy and reliability of service provided to customers. Water utilities want to provide a safe, reliable supply of water at a reasonable level of service (and reasonable cost). A reasonable level of service can be defined in many ways, but it should generally include provisions for adequate pressure, fire protection, and reliability of supply:

- Adequate pressure can be defined in terms of the minimum pressure under specific consumption conditions. Water systems are commonly designed to provide adequate pressure during maximum hour or maximum day plus fire flow conditions.
- Adequate fire protection refers to providing adequate flow to meet specific flow requirements for fire fighting.
- Reliability refers to the consistency of supply with which water is delivered. Redundancy is provided by looping of water mains, extra pumps, additional reservoirs, and backup sources of supply. Looping refers to providing a second feed to an area so that if one supply source is out of service, the other will still be available.

The evaluation criteria defining the level of service for the water system are presented in Section 5 (Review of Water Planning and Design Criteria) of this technical memorandum.

3.0. Review of Hydraulic Model Tool

A functional, calibrated model was used to assess the performance of DOU's water distribution and transmission system. The hydraulic model can be used to better understand and assess the capabilities of the DOU's system by simulating and identifying hydraulic limitations – low pressures and fire flow limitations – within the system under specified demand conditions. It is important to note that the model was calibrated using conditions that occurred during field testing in April 2003. Calibration is best when demand conditions with varying intensity and duration are used. By using a variety of demand conditions, the response of the system under critical flow conditions can be tested and the level of confidence in the model results can be assessed.

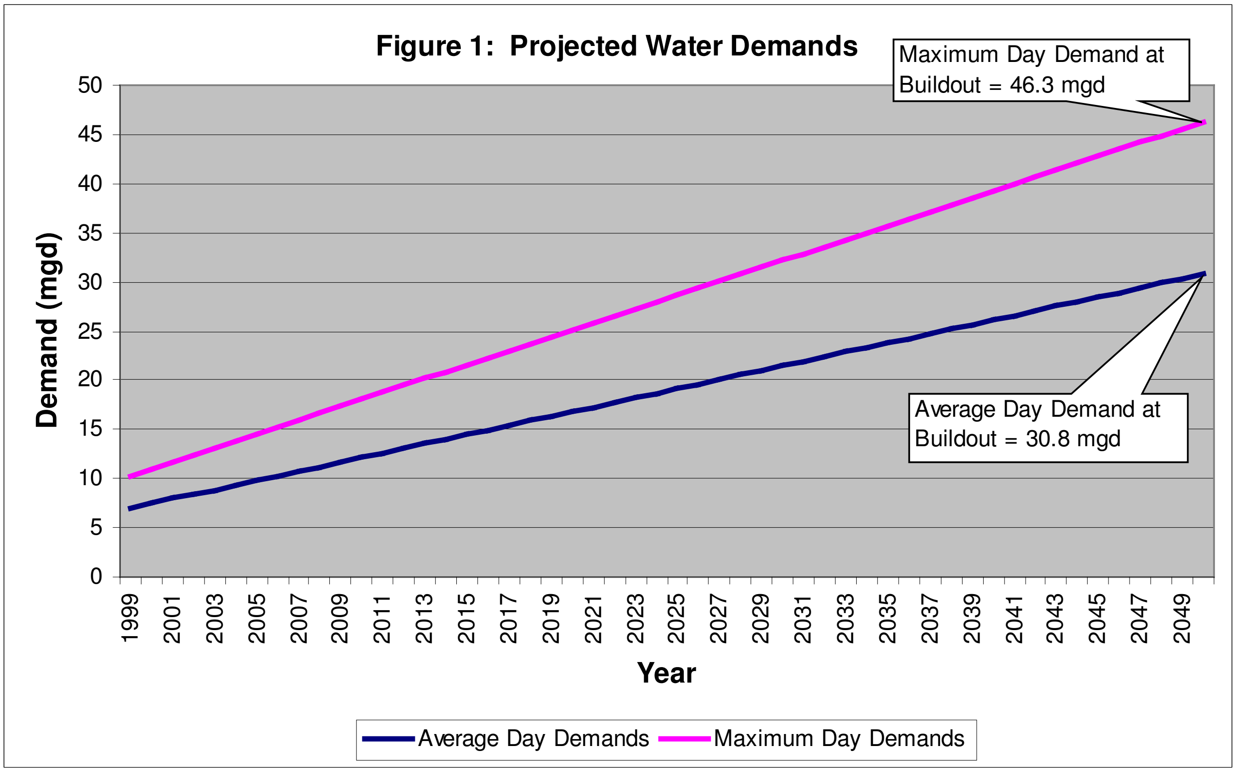
The hydraulic model will be a very valuable tool for DOU provided that the input files are maintained and updated as the distribution and transmission system expands and changes. This includes collecting additional data on demand conditions with varying characteristics. When used in conjunction with the other tools, such as GIS, SCADA, the model will serve as an integral part to the successful management and operation of the DOU distribution and transmission system.

A detailed discussion of model calibration is presented in Technical Memorandum 4 (*Development and Calibration of H2OMAP Water Hydraulic Model*).

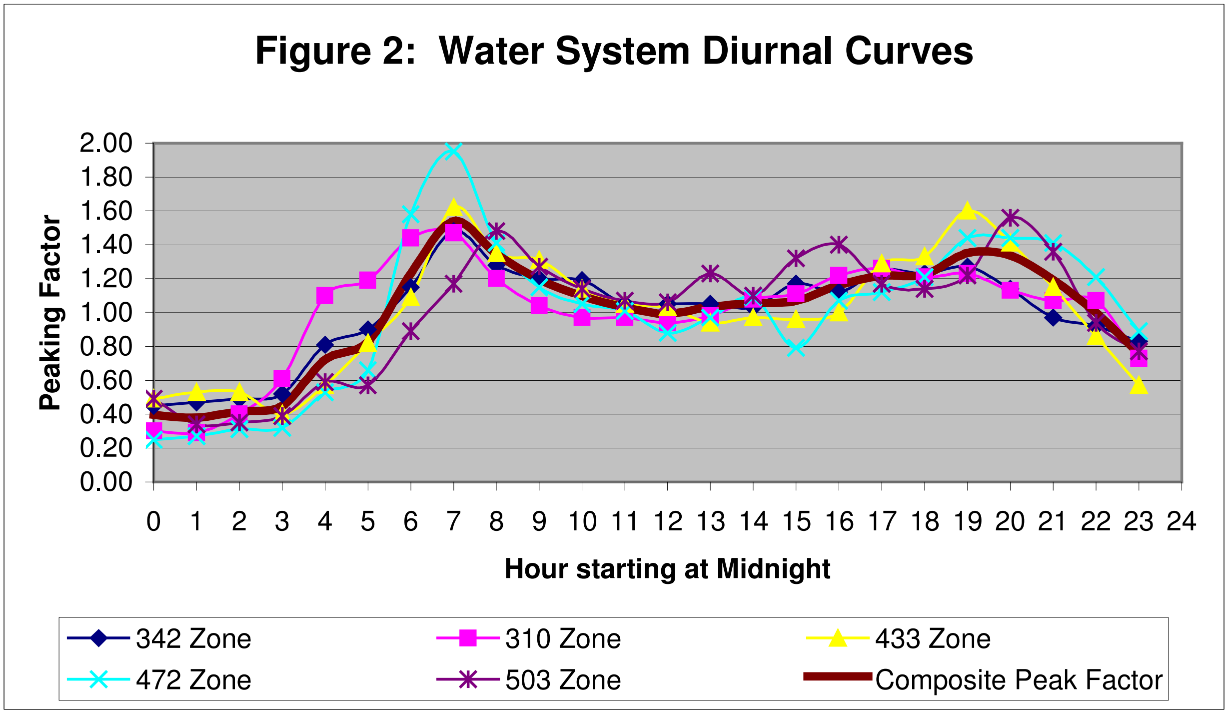
4.0. Review of Water Demands and Nodal Allocation

Water demands represent the average flows that are applied to the water system network from the contributing area. These demands are defined as the amount of water that must be carried by the distribution system to satisfy the need. Nodes represent points in the water system where water demands are taken from the system. For the model of the existing system that was used for calibration, DOU provided the water demands based on customer billing data for 2001. This approach results in an accurate allocation of water demands for model calibration.

Future water demands were projected using the estimated consumption method described in Technical Memorandum 2 (*Water Demands*). This method uses land use, customer class consumption values, and consumption ratios (diurnal demand curves) to determine the maximum day and peak hour demand conditions. The average and maximum day demands through the planning period are shown in Figure 1.



Demands in water systems vary throughout the day with peaks in the morning and evening and low flows in the early morning hours. Patterns are used to represent the daily temporal variations within the water system. They consist of a collection of multipliers (multiplication factors) that are applied to the daily demand to allow it to vary over time during an extended period simulation (EPS). Different patterns can be applied to individual nodes or groups of nodes to accurately represent water duties (e.g., residential, commercial, etc.). For the calibration analysis, DOU developed diurnal demand curves for each of the five pressure zones based on monitoring data collected over a period of several days. The diurnal curves used in this Master Plan are shown in Figure 2. The diurnal curves used for modeling each pressure zone are based on combined demand categories (i.e., separate diurnal curves for various land use types such as residential and commercial were not generated). The diurnal curves were based on average hourly factors (pattern timestep in model) over a 24-hour period (duration in model). The diurnal demand curve was considered to be uniform throughout the pressure zone. Consequently, average daily water demands at nodes in each pressure zone were multiplied by their respective diurnal demand curve to generate daily variations in water demand.



The general process for estimating the consumption at each node included:

- Establishing the base map of the service area. It should be noted that water service can be extended to the entire County and service outside the limits of the Urban Service Area is allowed, but is generally limited to groundwater well failures.
- Obtain the land use areas and customer class assignments based on the Land Use Plan.
- Calculate the average day demand defined by land use customer classes as described in Technical Memorandum 2 (*Water Demands*).
- Overlay the map of land use customer classes and the nodes in the water model.
- Establish the area of influence for each node. Node areas of influence establish which demands will be assigned to which nodes. Generally, a line that (1) is perpendicular to the line connecting two nodes, and (2) intersects that line at its midpoint, will be used to determine the closest node to which the demand can be assigned (demand polygon).
- Sum up the demands within each node's area of influence (demand polygon).
- Generate diurnal demand curves for individual pressure zones that are applied to the demands at each node.

This technique for assigning water demands to nodes in the model can easily accommodate changes in water duties for land uses and reconfiguration of the model network.

5.0. Review of Water Planning and Design Criteria

The performance of a finished water distribution system is judged by its ability to deliver the required flows while maintaining desirable pressure and water quality. Customer water demands and fire flow requirements must be met. Meeting these requirements depends upon the proper design and performance of distribution and transmission piping, elevated and ground storage tanks, and high service and booster pumping stations.

5.1. Evaluation Criteria

The criteria presented in this section for pumping, storage and piping are generally used in evaluating water distribution systems. A comparative review of the Water and Sewer Planning and Design Criteria proposed for use in the DOU's Water and Sewer Master Plan project was performed to determine whether the criteria proposed are reasonable. The DOU's planning and design criteria were used to evaluate the DOU's existing water system and to plan future improvements, upgrades, and expansions of facilities.

Planning and design guidelines vary from state to state and from utility to utility. While national organizations, such as the American Water Works Association (AWWA), provide some guidelines and many states regulate certain performance criteria, planning and design criteria are often left to the discretion of the water utility. The planning and design criteria proposed for use in the DOU's Water and Sewer Master Plan project were compared with the criteria used by similar utilities in the region (e.g., location, estimated population served, growth rate, customer demographic, etc.). It is important to recognize that the planning and design criteria should be applied on a case-by-case basis and may change over time.

DOU's planning and design criteria for waterworks facilities is summarized as follows:

- Water treatment facilities shall be adequate to provide the maximum day water demand.
- Water booster pumping stations shall be adequate to pump the maximum day water demand. While pumping stations are typically sized for maximum day demands, it may be desirable to size pumping facilities for peak hour demands (or a portion of peak hour demands) if the pumping station serves a pressure zone with a single storage tank that must be taken out-of-service for maintenance. It is generally desirable to provide at least two storage tanks per pressure zone to simplify operation of the pumping facilities when a tank is taken out-of-service. The Virginia Department of Health's (VDH) "Waterworks Regulations" require that each pumping station shall have at least two pumping units. Pumps should have sufficient capacity so that if any one pump is out-of-service (firm capacity) the remaining units shall be capable of providing the maximum day demand.
- Pipelines are sized for the following:
  - The largest of maximum hour flow, maximum day flow plus fire flow, or replenishment flow. Fire flow requirements are a primary factor affecting the sizing of piping in the water distribution system (6-inch and 8-inch mains).
  - An allowable velocity of 5 ft/sec.
  - An allowable headloss of 2-5 feet/1,000 feet of pipeline.
- Maximum pressure refers to the maximum pressure that the customer will experience. It is often in the range of 90-110 psi. The most common maximum design pressure among utilities is 100 psi; however, 80 psi is used in some building codes. The Uniform Plumbing Code requires that water pressures not exceed 80 psi at service connections, unless the service is provided with a pressure reducing device. This pressure is based on common household appliance limitations (water heaters can withstand 120-130 psi). The maximum pressure will occur when there is little head loss in the system (i.e., static pressure when tanks are full). Maximum water pressures at the service connections were set at 120 psi.
- Minimum pressure is the minimum pressure at a customer's tap. The most common minimum pressure among utilities is 40 psi. If pressures are less than 40 psi, there could be a noticeable pressure decrease when more than one device (e.g., faucet, toilet, shower, etc.) is used. The Virginia Department of Health's Waterworks Regulations require that the water system shall provide a minimum pressure of 20 psi at the service connection based on the greater of maximum hour or maximum day plus fire flow demand condition.

- Pressure fluctuation is the difference between maximum hour and minimum hour conditions at any one location in the system. An acceptable pressure fluctuation is 20-30 psi. Customers come to rely on steady pressure; thus in the interest of providing good service, large pressure fluctuations should be avoided in design. The maximum pressure fluctuation for this Master Plan was 30 psi. It is important to recognize that the pressures identified in the water model at a node may not accurately represent the pressures at the service connections due to the difference in elevation between the model node and the service connection point. Consequently, it may be necessary to review the model results more closely in areas near the pressure thresholds.
- Pressure zone layout refers to the design and layout of pressure zones across the system. Because pressure is related to ground elevation, a system covering hilly or mountainous terrain will have more pressure zones than one covering relatively flat terrain. The minimum pressure establishes the highest ground elevation that can be supplied, and the maximum pressure establishes the lowest ground elevation. Pressure zone boundaries can be moved to increase or decrease pressures and resolve pressure complaints from customers in the vicinity of the boundaries.
- Pressure regulating valves are classified as either pressure reducing or pressure sustaining valves. Pressure reducing valves are designed to maintain a constant downstream pressure regardless of the flow of water. They are often used at pressure zone boundaries when a source of water at a desired hydraulic grade line is needed. The desired maximum and minimum flow, pressure drop across the valve, and water velocity are criteria used to determine the size of valve required. The minimum pressure differential to operate a pressure reducing valve is 10 psi for small valves (6-inch and smaller) and 5 psi for large valves (8-inch and larger). The maximum velocity allowed through the valve is typically 15-20 feet/sec.
- Looping refers to providing supply to a single point or an area through two or more pipelines. This practice provides a higher level of reliability (i.e., if one source is out-of-service to the area, supply can be provided from a second source).
- Pipe materials generally accepted include ductile iron, steel, concrete, and polyvinyl chloride (plastic or PVC). PVC is usually used for smaller diameter piping.

5.2. Distribution System Storage Facilities

Storage facilities must be sized to provide equalization, fire and emergency storage. Each of these components and other storage facility considerations are described in the following section.

5.2.1. Equalization storage

Equalization storage is the amount of storage required to meet water demands in excess of the system delivery capability. The intent of equalization storage is to make up the difference between the consumers' peak demand and the system's available supply. It is the amount of desirable stored water to accommodate fluctuations in demand so that extreme variations in flow will not be imposed on the supply facilities.

The amount of equalization storage required is a function of the high service pumping capacity at the water treatment plant, distribution system capacity, and system demand characteristics. Equalization storage is generally less expensive than increased pumping capacity (including additional treatment capacity) and transmission and distribution system piping beyond that required to meet the maximum day demand (MDD). Consequently, it is desirable to size the pumping and piping systems to carry MDD with equalization storage sized to carry demands in excess of the MDD up to the peak hour demand (PHD). According to *Distribution Network Analysis for Water Utilities* (AWWA), equalization storage should represent approximately 50 percent of the total storage required. This guideline was achieved by maintaining at least 50 percent of the volume of the individual storage tanks during extended period simulation (EPS) modeling runs conducted under current and future conditions. Plots showing the fluctuation in tank levels over a 24-hour period were used to assess the volume of equalizing storage available.

5.2.2. Fire Storage

Fire flows have four characteristics: flow, duration, residual pressure, and looping. The volume of fire storage needed is primarily dependent on flow and duration:

- Flow is defined in terms of flowrate (typically gallons per minute) and can vary from 750 gpm for single family housing to 10,000 gpm for shopping malls. It is generally assumed that a major fire will not occur during maximum hour because the chance of this happening is so small. However, it is more likely that a fire would occur on maximum day so fire flow rates are usually imposed on maximum day demand. The County uses a fire flow rate of 2,500 gpm.
- Duration of the fire generally ranges from two hours to eight hours and is important in the planning and design of new storage facilities because it affects the sizing. The County uses a fire flow duration of three hours.

Fire storage requirements are typically dependent on the ISO requirements. The ISO determines fire flow requirements throughout the service area based on the characteristics of the individual buildings (structures). A comparison of DOU's fire flow requirements and typical requirements is shown in Table 5.

Table 5. Fire flow requirements

Source	Land Use		
	Residential	Commercial	Industrial
Stafford County	**1000 – 2500 gpm**	**2500 gpm**	**2500 gpm**
Loudoun County Sanitation Authority, VA	1000 – 2000 gpm	3000 gpm	3000 gpm
Anne Arundel County, MD	750 – 2500 gpm	3000 gpm	3000 gpm
Rivanna Water & Sewer Authority, VA	750 – 2000 gpm	2000 gpm	2000 gpm

As shown in Table 5, the fire flow requirements proposed by DOU are comparable to the fire flow guidelines used by nearby water utilities.

Each system storage facility should have reserves of fire storage equal to the amount required to furnish fire flow requirements within the area of influence for the individual storage facility. The area of influence is a function of area water consumption demands, fire flow demands and distribution system piping. For a large fire flow demand (in excess of 3000 gpm), more than one storage facility may be necessary to overcome limitations in piping or other distribution features. In some cases, smaller fire flow demands may be met by more than one facility due to particular features of the distribution system.

Steady-state modeling runs were performed under maximum day demand plus fire flow conditions to assess fire flow availability at each node at a minimum system pressure of 20 psi. Plots showing the fire flow availability at each node were used to assess the need for system improvements. The required fire flow should be specified for each node based on the type of land use served by the node (i.e., residential, commercial, etc.). For this study, the required fire flow was based on DOU's knowledge of the existing and proposed land use within the water system. Nodes that have deficient fire flow based on modeling can be field tested or reviewed to identify whether reduced fire flow rates are acceptable. Correcting fire flow deficiencies by replacing smaller piping with larger mains could result in longer water age and potential water quality problems.

5.2.3. Emergency Storage

Emergency storage is required to provide water during emergencies such as pipeline failures, main breaks, equipment failures, electrical power outages, water treatment facility failures, or natural disasters. The most likely emergency is a power failure lasting several hours or a trunk main failure, either of which would limit distribution capacity in a localized area. The DOU service area also could be subjected to a major disaster such as a hurricane, tornado or extended flooding. However, it is not economically feasible to provide sufficient emergency storage to accommodate emergency circumstances as severe as a hurricane or extended flooding.

The amount of emergency storage included within a particular water distribution system is an owner's option based on an assessment of risk and a capability to pay for the standby provisions. Unlike equalization and fire storage, which should generally be at all system storage sites, emergency storage may be included at only one or a limited number of storage sites.

5.2.4. Clearwell Storage

In addition to using different parameters to set the storage allocation, these parameters are often used in different ways (e.g., many utilities choose to determine equalization storage volume for each individual pressure zone in the system, some utilities choose to include clearwell storage at the treatment facilities, etc.).

Clearwell storage duplicates the function of system storage in that it compensates for system demands in excess of the water treatment plant capacity and allows a more stable rate of water treatment plant operation. For purposes of this distribution system analysis, the volume of clearwell storage is considered to be minimal and is not included in the storage assessment.

5.2.5. Impact of System Storage on Water Quality

The guidelines presented in this technical memorandum for sizing distribution system storage are intended to meet fire flow requirements and provide equalization and emergency storage. Excess storage or low turnover in storage tanks impacts water quality adversely by increasing residence time in the system, which may result in the following:

- low disinfectant residual
- higher disinfection byproducts
- bacterial regrowth

There is a need to balance storage requirements with water quality. In general, storage for fire protection and flow equalization should not be modified from the required or recommended amounts. Water quality in the distribution system can be optimized by:

- Optimizing operation of existing storage facilities (increasing tank turnover).
- Optimizing operation of the distribution system and pressure zones.
- Design of emergency or reserve storage in new storage facilities.

5.2.6. Storage Summary

Storage allocation parameters used by Stafford County, industry standards and guidelines, and local utilities are presented in Table 6.

Table 6. Storage allocation parameters

Source	Total Storage	Equalizing Storage	Fire Storage	Emergency Storage
Stafford County	***50% of AD***	***20% of MD***	***2500 gpm: 3 hrs***	***25% of total***
Distribution Network Analysis for Water Utilities, (M32), AWWA, 1989	40-50% of AD	20-25% of AD	Up to 2500 gpm: 2 hrs 3000-2500 gpm: 3 hrs Greater than 3500 gpm: 4 hrs	Not defined.
Distribution System Requirements for Fire Protection (M31), AWWA, 1989	Not defined.	30-40% of total	70-75% of total	
Virginia Department of Health Waterworks Regulations, 1997	50% of AD (derived from 200 gpd/EDU)	Not defined.	Not defined.	Not defined.
Recommended Standards for Water Works (Ten States), 1997	100% of AD	Not defined.	Not defined.	Not defined.
Anne Arundel County, MD	Not defined.	20% of MD	1000 gpm: 2 hrs or 3000 gpm: 3 hrs	50% of AD
Fairfax County Water Authority	Not defined.	15% of MD	3000 gpm: 4 hrs	Not defined.
Washington Suburban Sanitary Commission	Sum of individual components	17-20% of MD	2500 gpm: 2hrs up to 9500 gpm: 9 hrs	Unsatisfied MD demand: 4 hrs

After examining Table 6, several conclusions can be drawn:

- Total storage is typically 50-100% of the average day demand.
- Equalizing storage is frequently taken to be one-half of the total storage and can be determined based on a percentage of average day demand or maximum day demand. The volume of equalizing storage required can also be determined by performing a mass balance on the peak day.
- Fire and emergency storage is often considered to be one-half of the total storage (i.e., one-half equalizing storage and one-half fire and emergency storage). Fire storage is generally based on a flow rate over a given period. The County's fire storage criteria of 2500 gpm for 3 hours appears to be conservative for the overall system and it may be possible to reduce this volume based on a case-by-case review of each pressure zone. The volume of emergency storage varies considerably and is often taken to be a percentage of the total storage.

AWWA provides suggestions for sizing distribution storage facilities, but does not currently publish recommendations. According to the VDH "Waterworks Regulations", the minimum acceptable effective finished water storage for domestic purposes must be greater than 200 gallons per equivalent residential connection at minimum pressure (this essentially equates to one-half of the annual average day demand). Effective storage for each storage facility is determined by establishing the level in each tank above which all points in the water system can be served at 20 psi or higher (based on peak hour or maximum day plus fire flow). For this Master Plan, the total volume of storage needed will be equal to one-half of the annual average day demand and EPS runs will be performed to verify proper drawdown and refill of storage facilities.

6.0. Recommended Water System Improvements

This section identifies the major components of DOU's water distribution system and evaluates the performance and operation of the system compared with the criteria presented in Section 5. The evaluation is based on a review of existing operational data, discussions with DOU staff, and results of the simulations from the H2OMAP Water modeling. The water system improvements presented in this section are shown on the figure in the pocket at the end of this Master Plan (Proposed Water System Improvements) and the schedule showing the timing for implementation of the improvements is included in the pocket at the end of this Master Plan (Summary of Costs and Schedule for Recommended CIP Improvements (inside Urban Service Area)).

The water system improvements presented in this Master Plan are for the area inside the Urban Service Area. As shown on the Proposed Water System Improvements figure in the back pocket at the end of this Master Plan, the piping needed to meet the water demands for the area outside the Urban Service Area was evaluated in order to properly size the transmission system needed to deliver flows to the outer areas of the County. The County recently adopted a policy that water service can be extended to the entire County and service outside the limits of the Urban Service Area is allowed, but is generally limited to groundwater well failures. Constructing large diameter pipes (12-inch and above) in the rural areas of the County to meet fire flows and other water system criteria could result in water quality degradation in the piping network in the rural areas due to long residence times caused by long lengths of large diameter pipe with low water demands. Consequently, careful planning will be needed if the water system is expanded to these rural areas outside the Urban Service Area.

6.1. Summary of Storage Requirements

Tables 7 and 8 summarize the storage requirements by pressure zone for near-term and buildout conditions.

Table 7: Water distribution system storage adequacy (near-term conditions)

Zone	Average Day Demand (MGD)	Required Storage at 50% of Average Day Demand (MG)	Existing Storage at Near-term (MG)	Storage Deficit (-) or Surplus (+) at Near-term (MG)	Additional Storage Proposed at Near-term (MG)
310	2.3	1.15	Stone River - 2 Midway - 0.2 Moncure - 0.75 Smith Lake - 3.22	5.02	0
320	1.1	0.55	Ferry Road - 1.0	0.45	0
342	2.6	1.30	Abel Lake (pumped) – 4 (3.55 MG of storage remaining in Abel Lake Tank after 0.45 MG allocation to 503 Zone)	2.25	1.0 MG elevated at Grafton Tank
370N	2.1	1.05	0	-1.05	1.0 MG elevated at Courthouse Tank 0.5 MG elevated at Embrey Mill
370S	0.5	0.25	Berea – 0.5 (0.25 MG of remaining storage in Berea Tank after 503 Zone allocation)	0	0
433	2.4	1.20	Amyclae - 1.5 Shelton Shop – 0.36 *	0.66	0
472	0.8	0.40	Vista Woods - 0.5	0.10	0
503	1.4	0.70	Berea – 0.5 (0.25 MG of remaining storage in Berea Tank after 370S Zone allocation)	-0.45	0.45 MG from 2.7 MG surplus at Abel Lake Tank via pumping through Berea PS
Total	13.2	6.6	13.58	6.98	2.95

* Based on top 25 feet of standpipe (total height = 95 feet).

Tanks Replaced at Near-term
Courthouse - 0.25 MG
Grafton - 0.15 MG

Tanks Removed at Near-term
Bandy - 0.15 MG
Cranes Corner – 0.2 MG

Table 8: Water distribution system storage adequacy (buildout conditions)

Zone	Average Day Demand (MGD)	Required Storage at 50% of Average Day Demand (MG)	Existing Storage at Buildout (MG)	Storage Deficit (-) or Surplus (+) at Buildout (MG)	Additional Storage Proposed at Buildout (MG)
310	5.5	2.75	Stone River - 2 Midway – 0.2 Moncure – 0.75 Smith Lake - 3.22	3.42	0
320	2.3	1.15	Ferry Road - 1.0	-0.15	0.5 MG elevated at Sherwood Forest Farm Road
342	6.2	3.10	0	-3.10	1.0 MG elevated at Grafton Tank 1.0 MG elevated at Cranes Corner 1.5 MG from 2.0 MG elevated near Abel Lake WTP
370N	4.0	2.00	0	-2.00	1.0 MG elevated at Courthouse Tank 0.5 MG elevated at Embrey Mill 0.5 MG from 2.0 MG elevated near Abel Lake WTP
370S	1.1	0.55	0	-0.55	0.5 MG from 1.0 MG elevated at Greenbank Road through PRV
433	4.9	2.45	Amyclae - 1.5	-0.95	1.0 MG elevated at Shelton Shop
472	1.8	0.90	Vista Woods - 0.5	-0.40	0.5 MG elevated along Garrisonville Road
480	2.9	1.45	0	-1.45	0.5 MG from 1.0 MG elevated at Greenbank Road 1 MG Pumped from 2 MG at Rocky Pen Run WTP
520	2.3	1.15	0	-1.15	1.0 MG elevated at Clark Patton Road 0.15 MG from 2 MG Pumped at Rocky Pen Run WTP
Total	31.0	15.5	9.17	-6.33	10.65

<u>Tanks Replaced Prior to Buildout</u>
Courthouse – 0.25 MG
Grafton - 0.15 MG
Shelton Shop Standpipe – 1.375 MG

<u>Tanks Removed Prior to Buildout</u>
Bandy - 0.15 MG
Cranes Corner - 0.2 MG
Abel Lake - 4 MG
Berea – 0.5 MG

6.2. Finished Water Pumping

Table 9 summarizes the finished water pumping facilities in the DOU distribution system.

Table 9: Water pumping stations

Pumping station	Existing Capacity (mgd)	Near-term Capacity (mgd)	Buildout Capacity (mgd)
Smith Lake	10	14	14
Moncure	5.0	8.5	8.5
Vista Woods	1.0	1.6	1.6
Abel Lake	6.0	6.0	6.0
Berea	3.2	3.2	Emergency backup
Cranes Corner	5.2	5.2	Emergency backup
Mountain View	Emergency backup	Emergency backup	1.6
Embrey Mill	Not applicable	2.0	Emergency backup
370N Zone	Not applicable	Not applicable	10
433 Zone	Not Applicable	Not Applicable	2.8
472 Zone	Not Applicable	Not Applicable	1.1
520 Zone	Not Applicable	Not Applicable	2.8

6.3. 310 Zone Improvements

310-01: Construct 8-inch main from Jib Drive to Hope Springs Lane (1,963 feet)

This project includes design and construction of an 8-inch water main from Jib Drive to Hope Springs Lane (1,963 feet). The purpose of the project is to improve fire flows and enhance reliability to customers in the vicinity of Hope Springs Lane that are served by a single 6-inch main and Walker Way and Jib Drive that are currently served by a single 8-inch water main. This project is independent of other proposed water system improvements in the 310 Zone and the timing for implementation is driven by the need to increase fire flow capabilities or reliability in this area.

Priority	*2 - Necessary*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Total Project Cost	*$127,000*
Prior Spending	*$0*
Costs in this Plan Period	*$127,000*

310-03: Construct 12-inch main along Jefferson Davis Highway from Sunnyside Drive to Slake Drive (662 feet)

This project includes design and construction of a 12-inch water main along Jefferson Davis Highway from Sunnyside Drive to Slake Drive (662 feet). The purpose of the project is to connect the 12-inch mains along Jefferson Davis Highway to improve flows from Smith Lake WTP to customers along the Jefferson Davis Highway corridor.

Priority	*2 - Necessary*
Design	*2004*
Construct	*2005*
Total Project Cost	*$57,000*
Prior Spending	*$0*
Costs in this Plan Period	*$57,000*

310-04: Construct 12-inch main from Brittany Lane to Lafayette Street (1,205 feet)

This project includes design and construction of a 12-inch water main from Brittany Lane to Layfayette Street (1,205 feet). The purpose of the project is to connect the 12-inch mains along Brittany Lane and Layfayette Street to increase transmission capacity from Smith Lake WTP to the Embrey Mill Pumping Station and enhance system reliability. Currently, a single 12-inch main along Mine Road serves the Austin Ridge and the Embrey Mill PS area. This project is recommended for implementation in the near-term (i.e., prior to Rocky Pen Run WTP) to provide a second 12-inch main for transferring flows from Smith Lake WTP to the Embrey Mill PS and ultimately through the 370N Zone to the pressure zones in the southern portion of the County.

Priority	*2 - Necessary*
Design	*2007*
Construct	*2008*
Total Project Cost	*$109,000*
Prior Spending	*$0*
Costs in this Plan Period	*$109,000*

310-05: Construct 12-inch main along Aquia Drive from Coal Landing Road to Washington Drive (4,146 feet)
This project includes design and construction of a 12-inch water main along Aquia Drive from Coal Landing Road to Washington Drive (4,146 feet). The purpose of the project is to increase transmission capacity between Smith Lake WTP and the Stone River Tank to improve operation of the tank (i.e., drawdown and refill characteristics), as well as enhance reliability in Aquia Harbour and the southeastern portion of the 310 Zone. The timing for implementation is based on improving operation of the Stone River Tank. As demands in the vicinity of the Stone River Tank increase through the planning period, the volume of water depleted from the Stone River Tank during high demand periods will increase requiring a larger quantity of water through the transmission system to replenish the depleted tank storage.

As an alternative to construction of the 12-inch main along Aquia Drive, the existing 12-inch main along the Jefferson Davis Highway could be replaced with a larger main to increase transmission capacity to the Stone River Tank.

Priority	2 - Necessary
Design	*2019*
Construct	*2020*
Total Project Cost	*$356,000*
Prior Spending	*$0*
Costs in this Plan Period	*$356,000*

310-06: Construct 12-inch main along Washington Drive from Aquia Drive to Jefferson Davis Highway (5,830 feet)
This project includes design and construction of a 12-inch water main along Washington Drive from Aquia Drive to Jefferson Davis Highway (5,830 feet). The purpose of the project is to increase transmission capacity between Smith Lake WTP and the Stone River Tank to improve operation of the tank (i.e., drawdown and refill characteristics), as well as enhance reliability in Aquia Harbour and the southeastern portion of the 310 Zone. The timing for implementation is based on improving operation of the Stone River Tank. As demands in the vicinity of the Stone River Tank increase through the planning period, the volume of water depleted from the Stone River Tank during high demand periods will increase requiring a larger quantity of water through the transmission system to replenish the depleted tank storage.

As an alternative to construction of the 12-inch main along Washington Drive, the existing 12-inch main along the Jefferson Davis Highway could be replaced with a larger main to increase transmission capacity to the Stone River Tank.

Priority	*2 - Necessary*
Design	*2018*
Construct	*2019*
Total Project Cost	*$501,000*
Prior Spending	*$0*
Costs in this Plan Period	*$501,000*

310-07: Construct 24-inch main along Garrisonville Road (Rt. 610) from Salisbury Drive to Jefferson Davis Highway (2,926 feet)
This project includes design and construction of a 24-inch water main along Garrisonville Road (Route 610) from Salisbury Drive to Jefferson Davis Highway (2,926 feet). The purpose of the project is to increase transmission capacity between Smith Lake WTP and the eastern portion of the 310 Zone,

increase flow to the Stone River Tank to improve operation of the tank, and enhance reliability in Aquia Harbour and the eastern portion of the 310 Zone. This project provides a strong second connection across I-95 from Smith Lake WTP to the piping in the eastern portion of the 310 Zone. The 24-inch main from the Smith Lake WTP to the Moncure PS serves as a strong feed for the proposed 24-inch main under I-95. The timing for this project is dictated by the need for increased transmission capacity due to higher demands during the planning period.

Priority	*2 - Necessary*
Design	*2017*
Construct	*2018*
Total Project Cost	*$798,000*
Prior Spending	*$0*
Costs in this Plan Period	*$798,000*

310-08: Replace existing 8-inch main along Coal Landing Road with a 12-inch main from Greenridge Drive east to existing 12-inch main (1,873 feet)

This project includes replacement of the existing 8-inch main along Coal Landing Road with a 12-inch main from Greenridge Drive east to the existing 12-inch main (1,873 feet). The purpose of the project is to increase conveyance capacity between Smith Lake WTP and the Stone River Tank to improve operation of the tank (i.e., tank drawdown and refill), as well as enhance reliability in Aquia Harbour and the southeastern portion of the 310 Zone. The timing for this project is dictated by the need to improve operation of the Stone River Tank for increased transmission capacity due to higher demands during the planning period. As demands in the vicinity of the Stone River Tank increase through the planning period, the volume of water depleted from the Stone River Tank during high demand periods will increase requiring a larger quantity of water through the transmission system to replenish the depleted tank storage.

Priority	*2 - Necessary*
Design	*2020*
Construct	*2021*
Total Project Cost	*$161,000*
Prior Spending	*$0*
Costs in this Plan Period	*$161,000*

310-10: Construct 24-inch main from I-95 to 12-inch main along Jefferson Davis Highway near Sunnyside Drive (2,197 Feet)

This project includes design and construction of a 24-inch water main from I-95 to the 12-inch main along Jefferson Davis Highway near Sunnyside Drive (2,197 feet). The purpose of the project is to increase transmission capacity from Smith Lake WTP to the 12-inch mains along Jefferson Davis Highway to improve flows to customers along the Jefferson Davis Highway corridor.

Priority	*3 – Prior Appropriation*
Design	*Not Applicable*
Construct	*Not Applicable*
Total Project Cost	*$680,000*
Prior Spending	*$0*
Costs in this Plan Period	*$680,000*

310-11: Construct 10-inch main along Jefferson Davis Highway from Terrace Drive (4,000 Feet)

This project includes design and construction of a 10-inch water main along Jefferson Davis Highway from Terrace Lane near Sunnyside Drive (4,000 feet). The purpose of the project is to increase conveyance capacity from the 12-inch main along Jefferson Davis Highway to the northern area of the 310 Zone between I-95 and the Jefferson Davis Highway.

Priority	*3 – Prior Appropriation*
Design	*Not Applicable*
Construct	*Not Applicable*
Total Project Cost	*$305,000*
Prior Spending	*$0*
Costs in this Plan Period	*$305,000*

310-200: Expand Smith Lake Pumping Station to 14 mgd

Smith Lake WTP currently supplies water to three pressure zones with hydraulic grade lines of 310, 433 and 472 feet through 30-inch and 24-inch water mains. Water from the Smith Lake WTP is pumped to the Moncure PS on the western border of the 310 Zone which pumps flow to the 433 Zone. Flow from the 433 Zone is boosted to the 472 Zone through the Vista Woods PS which is located on the western border of the 433 Zone along Shelton Shop Road. The 310 Zone has three tanks (Midway, Stone River and Moncure), the 433 Zone has two tanks (Shelton Shop and Amyclae), and the 472 Zone has one tank (Vista Woods).

This project involves expansion of the Smith Lake Pumping Station from 10 mgd to 14 mgd. The purpose of this project is to expand the pumping capacity to fully utilize the available treatment capacity from Smith Lake WTP and meet projected demands. Utilizing the available treatment capacity at Smith Lake WTP is particularly important in the near-term prior to Rocky Pen Run WTP being on-line. Comparing projected water demands and the existing capacities of the pumping stations at Smith Lake and Abel Lake WTPs, the near-term demand is expected to reach the treatment capacity of 20 mgd (14 mgd from Smith Lake WTP and 6 mgd from Abel Lake WTP) by roughly 2013.

Priority	*1 - Critical*
Design	*2006*
Construct	*2007*
Total Project Cost	*$972,000*
Prior Spending	*$0*
Costs in this Plan Period	*$972,000*

310-300: Construct emergency pressure reducing valve between 370N/310 Zone near Wallace Lane

Three pressure reducing valves (PRVs) are proposed on transmission mains along the southern border of the 310 Zone to provide flow from the 370N Zone to the 310 Zone under emergency conditions that cause a disruption in service in the 310 Zone (e.g., major main breaks, Smith Lake WTP out-of-service, etc.). In the future, flow to the 370N Zone will be provided by both the Abel Lake and Rocky Pen Run WTPs while the 310 Zone will be served solely by Smith Lake WTP. Consequently, these PRVs significantly enhance system reliability by providing a second source of supply to the 310 Zone. The timing for construction of the PRVs is dictated by the establishment of the 370N Zone.

Priority	*2 - Necessary*
Design	*2021*
Construct	*2022*
Total Project Cost	*$65,000*
Prior Spending	*$0*
Costs in this Plan Period	*$65,000*

310-301: Construct emergency pressure reducing valve between 370N/310 Zone along Bells Hill Road near Byrum Street

Three pressure reducing valves (PRVs) are proposed on transmission mains along the southern border of the 310 Zone to provide flow from the 370N Zone to the 310 Zone under emergency conditions that cause a disruption in service in the 310 Zone (e.g., major main breaks, Smith Lake WTP out-of-service, etc.). In the future, flow to the 370N Zone will be provided by both the Abel Lake and Rocky Pen Run WTPs while the 310 Zone will be served solely by Smith Lake WTP. Consequently, these PRVs significantly enhance system reliability by providing a second source of supply to the 310 Zone. The timing for construction of the PRVs is dictated by the establishment of the 370N Zone.

Priority	*2 - Necessary*
Design	*2021*
Construct	*2022*
Total Project Cost	*$65,000*
Prior Spending	*$0*
Costs in this Plan Period	*$65,000*

310-302: Construct emergency pressure reducing valve between 370N/310 Zone along Olde Concord Road near Somerset Lane

Three pressure reducing valves (PRVs) are proposed on transmission mains along the southern border of the 310 Zone to provide flow from the 370N Zone to the 310 Zone under emergency conditions that cause a disruption in service in the 310 Zone (e.g., major main breaks, Smith Lake WTP out-of-service, etc.). In the future, flow to the 370N Zone will be provided by both the Abel Lake and Rocky Pen Run WTPs while the 310 Zone will be served solely by Smith Lake WTP. Consequently, these PRVs significantly enhance system reliability by providing a second source of supply to the 310 Zone. The timing for construction of the PRVs is dictated by the establishment of the 370N Zone.

Priority	*2 - Necessary*
Design	*2021*
Construct	*2022*
Total Project Cost	*$65,000*
Prior Spending	*$0*
Costs in this Plan Period	*$65,000*

6.4. 320 Zone Improvements

320-01: Construct 12-inch main along Kings Highway from Ferry Road to 12-inch main on Cool Springs Road (1,539 feet)

This project involves design and construction of a 12-inch main along Kings Highway from Ferry Road to the 12-inch main on Cool Springs Road (1,539 feet). The purpose of the project is to provide a connection between the existing 12-inch mains to create a stronger connection with the piping on Ferry Road and enhance flow south through the existing 12-inch main on Kings Highway. Construction of the water main should be concurrent with construction of the 342/320 Zone PRVs (320-300 and 320-301) and establishment of the 320 Zone which is driven by replacement of the existing Grafton Tank (342-100).

Priority	*2 - Necessary*
Design	*2005*
Construct	*2006*
Total Project Cost	*$132,000*
Prior Spending	*$0*
Costs in this Plan Period	*$132,000*

320-02: Construct 16-inch main from Kings Highway to 320 Zone elevated tank (3,219 feet)

This project involves design and construction of a 16-inch main from the existing 12-inch main along Kings Highway to the proposed water storage tank along Sherwood Forest Farm Road (3,219 feet). Typically, connecting pipes serving water tanks are 16-inch or larger. A 16-inch connecting pipe is proposed to provide flow to the 12-inch north and south of the connection on Kings Highway. The length of the water main serving the storage tank will be dependent on the location of the storage tank as determined by future siting studies. Construction of the water main will be concurrent with the water storage tank.

Priority	*2 – Necessary*
Design	*2024*
Construct	*2025*
Total Project Cost	*$344,000*
Prior Spending	*$0*
Costs in this Plan Period	*$344,000*

320-100: Construct 0.5 MG elevated storage tank along Kings Highway in vicinity of Sherwood Forest Farm Road

Abel Lake WTP currently supplies water to two pressure zones with hydraulic grade lines of 342 feet and 503 feet. Water from the Abel Lake WTP is pumped through a 16-inch water main to the Abel Lake Tank. The Abel Lake Tank is a 4 MG ground level tank with an overflow elevation of 298 feet. Water is pumped from the Abel Lake Tank to the 342 Zone through the Cranes Corner Pumping Station and to the 503 Zone through the Berea Pumping Station. Four elevated storage tanks are located in the 342 Zone:

- Cranes Corner (0.2 MG, 342 ft OF)
- Grafton (0.15 MG, 316 ft OF)
- Ferry Road (1 MG, 320 OF)
- Bandy (0.15 MG, 341 ft OF)

The Cranes Corner and Bandy Tanks are located near the Abel Tank/Cranes Corner Pumping Station while the Grafton and Ferry Road Tanks are distant from the supply. Currently, the Cranes Corner Pumping Station is operated off of the water levels in the Cranes Corner Tank and the pumps in the Cranes Corner Pumping Station cycle on/off due to the small volume of storage in the Cranes Corner Tank. After Rocky Pen Run WTP and the 342 Zone transmission main (30-inch) along Warrenton Road is complete, the 342 Zone will be fed from the vicinity of Warrenton Road and I-95 and the Cranes Corner Pumping Station will be eliminated along with the Abel Tank. The existing Grafton Tank needs to be repainted and was recommended for replacement due to its small size and inability to maintain adequate pressures in the area caused by its low overflow elevation (316 feet) and piping constraints in the area. Replacing the Grafton Tank with an elevated tank at a higher overflow elevation will cause the Cranes Corner and Bandy Tanks to typically remain full and not operate properly. Consequently, the small tanks at Cranes Corner (0.2 MG) and Bandy (0.15 MG) will be eliminated following construction of the new Grafton Tank. To improve low pressures in the vicinity of the Grafton Tank, the proposed Grafton Tank will be raised from an overflow elevation of 316 feet to 342 feet. The proposed Grafton Tank will be used to control the pumps at Rocky Pen Run Reservoir WTP that feed the 342 Zone.

The Ferry Road Tank, which is in proximity of the proposed Grafton Tank, has an overflow level of 320 feet and will be converted to 320 Zone service. The 320 Zone will be established to maintain acceptable operating pressures at the lower elevations in the southern portion of the existing 342 Zone. The 320 Zone will be established by closing interconnecting piping in the vicinity of White Oak Road and installing two pressure reducing valves on the 12-inch main along Butler Road near Cool Springs Road and on the 12-inch main along White Oak Road near Ferry Road.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 320 Zone will be 2.3 mgd. Consequently, the volume of storage needed in the 320 Zone under buildout conditions is roughly 1.1 MG. The existing Ferry Road Tank (1.0 MG) satisfies the projected storage requirements. In addition to the existing Ferry Road Tank, a second 0.5 MG tank (OF 320 ft) is proposed in the southern portion of the 320 Zone along Kings Highway in the vicinity of Sherwood Forest Farm Road. The southern portion of the 320 Zone is served by a single 12-inch main along Kings Highway. The purpose of this tank is to enhance reliability and provide adequate fire flows (2,500 gpm) in the vicinity of the tank. Alternatively, additional piping along Colebrook Road and McCarty Road could be constructed to create a looped network for the southern portion of the 320 Zone. An additional alternative considered included construction of a PRV at the 342/320 Zone border at McCarty Road and Forest Lane Road which assumed that piping was constructed in this area of the 342 Zone in the future. Based on hydraulic modeling, it was concluded that significant pressure reductions would occur in the 342 Zone in the vicinity of the PRV during the transfer of high flows to the 320 Zone (i.e., fire flows) through a PRV at Forest Lane Road.

Priority	*2 - Necessary*
Design	*2024*
Construct	*2025*
Total Project Cost	*$863,000*
Prior Spending	*$0*
Costs in this Plan Period	*$863,000*

320-300: Construct pressure reducing valve between 342/320 Zone along Butler Road near Cool Springs Road

Abel Lake WTP currently supplies water to two pressure zones with hydraulic grade lines of 342 feet and 503 feet. Water from the Abel Lake WTP is pumped through a 16-inch water main to the Abel Lake Tank. The Abel Lake Tank is a 4 MG ground level tank with an overflow elevation of 298 feet. Water is pumped from the Abel Lake Tank to the 342 Zone through the Cranes Corner Pumping Station and to the 503 Zone through the Berea Pumping Station. Four elevated storage tanks are located in the 342 Zone:

- Cranes Corner (0.2 MG, 342 ft OF)
- Grafton (0.15 MG, 316 ft OF)
- Ferry Road (1 MG, 320 OF)

- Bandy (0.15 MG, 341 ft OF)

The Cranes Corner and Bandy Tanks are located near the Abel Tank/Cranes Corner Pumping Station while the Grafton and Ferry Road Tanks are distant from the supply. Currently, the Cranes Corner Pumping Station is operated off of the water levels in the Cranes Corner Tank and the pumps in the Cranes Corner Pumping Station cycle on/off due to the small volume of storage in the Cranes Corner Tank. After Rocky Pen Run WTP and the 342 Zone transmission main (30-inch) along Warrenton Road is complete, the 342 Zone will be fed from the vicinity of Warrenton Road and I-95 and the Cranes Corner Pumping Station will be eliminated along with the Abel Tank. The existing Grafton Tank needs to be repainted and was considered for replacement due to its small size and inability to maintain adequate pressures in the area caused by its low overflow elevation (316 feet) and piping constraints in the area. Replacing the Grafton Tank with an elevated tank at a higher overflow elevation will cause the Cranes Corner and Bandy Tanks to typically remain full and not operate properly. Consequently, the small tanks at Cranes Corner (0.2 MG) and Bandy (0.15 MG) will be eliminated following construction of the new Grafton Tank. To improve low pressures in the vicinity of the Grafton Tank, the proposed Grafton Tank will be raised from an overflow elevation of 316 feet to 342 feet. The proposed Grafton Tank will be used to control the pumps at Rocky Pen Run Reservoir WTP that feed the 342 Zone.

The Ferry Road Tank, which is in proximity of the proposed Grafton Tank, has an overflow level of 320 feet and will be converted to 320 Zone service. The 320 Zone will be established to maintain acceptable operating pressures at the lower elevations in the southern portion of the existing 342 Zone. The 320 Zone will be established by closing interconnecting piping in the vicinity of White Oak Road and installing two pressure reducing valves on the 12-inch main along Butler Road near Cool Springs Road and on the 12-inch main along White Oak Road near Ferry Road.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 320 Zone will be 2.3 mgd. Consequently, the volume of storage needed in the 320 Zone under buildout conditions is roughly 1.1 MG. The existing Ferry Road Tank (1.0 MG) satisfies the projected storage requirements. In addition to the existing Ferry Road Tank, a second 0.5 MG tank (OF 320 ft) is proposed in the southern portion of the 320 Zone along Kings Highway in the vicinity of Sherwood Forest Farm Road. The southern portion of the 320 Zone is served by a single 12-inch main along Kings Highway. The purpose of this tank is enhance reliability and provide adequate fire flows (2,500 gpm) in the vicinity of the tank. Alternatively, additional piping along Colebrook Road and McCarty Road could be constructed to create a looped network for the southern portion of the 320 Zone. An additional alternative considered included construction of a PRV at the 342/320 Zone border at McCarty Road and Forest Lane Road which assumed that piping was constructed in this area of the 342 Zone in the future. Based on hydraulic modeling, it was concluded that significant pressure reductions would occur in the 342 Zone in the vicinity of the PRV during the transfer of high flows to the 320 Zone (i.e., fire flows) through a PRV at Forest Lane Road.

Priority	*2 - Necessary*
Design	*2005*
Construct	*2006*
Total Project Cost	*$65,000*
Prior Spending	*$0*
Costs in this Plan Period	*$65,000*

320-301: Construct pressure reducing valve between 342/320 Zone along White Oak Road near Ferry Road

Abel Lake WTP currently supplies water to two pressure zones with hydraulic grade lines of 342 feet and 503 feet. Water from the Abel Lake WTP is pumped through a 16-inch water main to the Abel Lake Tank. The Abel Lake Tank is a 4 MG ground level tank with an overflow elevation of 298 feet. Water is pumped from the Abel Lake Tank to the 342 Zone through the Cranes Corner Pumping Station and to the 503 Zone through the Berea Pumping Station. Four elevated storage tanks are located in the 342 Zone:

- Cranes Corner (0.2 MG, 342 ft OF)
- Grafton (0.15 MG, 316 ft OF)
- Ferry Road (1 MG, 320 OF)
- Bandy (0.15 MG, 341 ft OF)

The Cranes Corner and Bandy Tanks are located near the Abel Tank/Cranes Corner Pumping Station while the Grafton and Ferry Road Tanks are distant from the supply. Currently, the Cranes Corner Pumping Station is operated off of the water levels in the Cranes Corner Tank and the pumps in the Cranes Corner Pumping Station cycle on/off due to the small volume of storage in the Cranes Corner Tank. After Rocky Pen Run WTP and the 342 Zone transmission main (30-inch) along Warrenton Road is complete, the 342 Zone will be fed from the vicinity of Warrenton Road and I-95 and the Cranes Corner Pumping Station will be eliminated along with the Abel Tank. The existing Grafton Tank needs to be repainted and was considered for replacement due to its small size and inability to maintain adequate pressures in the area caused by its low overflow elevation (316 feet) and piping constraints in the area. Replacing the Grafton Tank with an elevated tank at a higher overflow elevation will cause the Cranes Corner and Bandy Tanks to typically remain full and not operate properly. Consequently, the small tanks at Cranes Corner (0.2 MG) and Bandy (0.15 MG) will be eliminated following construction of the new Grafton Tank. To improve low pressures in the vicinity of the Grafton Tank, the proposed Grafton Tank will be raised from an overflow elevation of 316 feet to 342 feet. The proposed Grafton Tank will be used to control the pumps at Rocky Pen Run Reservoir WTP that feed the 342 Zone.

The Ferry Road Tank, which is in proximity of the proposed Grafton Tank, has an overflow level of 320 feet and will be converted to 320 Zone service. The 320 Zone will be established to maintain acceptable operating pressures at the lower elevations in the southern portion of the existing 342 Zone. The 320 Zone will be established by closing interconnecting piping in the vicinity of White Oak Road and installing two pressure reducing valves on the 12-inch main along Butler Road near Cool Springs Road and on the 12-inch main along White Oak Road near Ferry Road.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 320 Zone will be 2.3 mgd. Consequently, the volume of storage needed in the 320 Zone under buildout conditions is roughly 1.1 MG. The existing Ferry Road Tank (1.0 MG) satisfies the projected storage requirements. In addition to the existing Ferry Road Tank, a second 0.5 MG tank is proposed in the southern portion of the 320 Zone along Kings Highway in the vicinity of Sherwood Forest Farm Road. The southern portion of the 320 Zone is served by a single 12-inch main along Kings Highway. The purpose of this tank is enhance reliability and provide adequate fire flows (2,500 gpm) in the vicinity of the tank. Alternatively, additional piping along Colebrook Road and McCarty Road could be constructed to create a looped network for the southern portion of the 320 Zone. An additional alternative considered included construction of a PRV at the 342/320 Zone border at McCarty Road and Forest Lane Road which assumed that piping was constructed in this area of the 342 Zone in the future. Based on hydraulic modeling, it was concluded that significant pressure reductions would occur in the 342 Zone in the vicinity of the PRV during the transfer of high flows to the 320 Zone (i.e., fire flows) through a PRV at Forest Lane Road.

Priority	*2 - Necessary*
Design	*2005*
Construct	*2006*
Total Project Cost	*$65,000*
Prior Spending	*$0*
Costs in this Plan Period	*$65,000*

6.5. 342 Zone Improvements

342-01: Construct 20-inch main along Warrenton Road from Olde Forge Road to Jefferson Davis Highway and Butler Road to Carter Street (7,217 feet)

This project involves design and construction of a 20-inch main along Warrenton Road from Olde Forge Road to Jefferson Davis Highway and Butler Road to Carter Street (7,217 feet). The purpose of the project is to convey flows from the 30-inch main connecting Rocky Pen Run WTP to the 342 and 320 Zones. A major problem in the near-term and future water system is the limiting transmission capacity from the western to eastern portions of the 342 Zone. This transmission main is a necessary feed for the 320 Zone as water demands increase through the planning period.

Priority	*2 - Necessary*
Design	*2019*
Construct	*2020*
Total Project Cost	*$947,000*
Prior Spending	*$0*
Costs in this Plan Period	*$947,000*

342-02: Construct 18-inch main along Butler Road from Carter Street to White Oak Road to Castle Rock Drive (4,614 feet)

This project involves design and construction of an 18-inch main along Butler Road from Carter Street to White Oak Road to Castle Rock Drive (4,614 feet). The purpose of the project is to convey flows to the southern portion of the 342 Zone and the 320 Zone. A major problem in the near-term and future water system is the limiting transmission capacity from the western to eastern portions of the 342 Zone. This project is proposed for the near-term to create a strong connection between the 12-inch mains in the vicinity of Jefferson Davis Highway with the 12-inch main along Cool Springs Road/Deacon Road.

Priority	*1 - Critical*
Design	*2006*
Construct	*2007*
Total Project Cost	*$538,000*
Prior Spending	*$0*
Costs in this Plan Period	*$538,000*

342-03: Construct 18-inch main along Butler Road from Castle Rock Drive to Deacon Road/Cool Springs Road intersection (2,443 feet)

This project involves design and construction of an 18-inch main along Butler Road from Castle Rock Drive to Deacon Road/Cool Springs Road intersection (2,443 feet). The purpose of the project is to convey flows to the southern portion of the 342 Zone and the 320 Zone. A major problem in the near-term and future water system is the limiting transmission capacity from the western to eastern portions of the 342 Zone. This project is proposed for the near-term to create a strong connection between the 12-inch mains in the vicinity of Jefferson Davis Highway with the 12-inch main along Cool Springs Road/Deacon Road.

Priority	*1 - Critical*
Design	*2006*
Construct	*2007*
Total Project Cost	*$285,000*
Prior Spending	*$0*
Costs in this Plan Period	*$285,000*

342-04: Construct 16-inch main along Forbes Street from Carter Street to Harrell Road (4,260 feet)

This project involves design and construction of a 16-inch main along Forbes Street from Carter Street to Harrell Road (4,260 feet). A major problem in the near-term and future water system is the limiting transmission capacity from the western to eastern portions of the 342 Zone. The purpose of the project is to convey flows to the southern portion of the 342 Zone by connecting the proposed 20-inch main along Butler Road with the 12-inch mains along Harrell and Deacon Roads.

Priority	*2 - Necessary*
Design	*2019*
Construct	*2020*
Total Project Cost	*$455,000*
Prior Spending	*$0*
Costs in this Plan Period	*$455,000*

342-05: Construct 24-inch main along RV Parkway and Beagle Road to Truslow Road (5,579 feet)

This project involves design and construction of a 24-inch main along RV Parkway and Beagle Road to Truslow Road (5,579 feet). The purpose of the project is to convey flows from the 30-inch main from Rocky Pen Run WTP to the 342 and 370N Zones. A major problem in the near-term and future water system is the limiting transmission capacity from the western to eastern portions of the 342 Zone. The project is proposed for the near-term to strengthen the connections between the existing 12-inch mains in order to convey flows from the Cranes Corner PS south and east through the 342 Zone.

Priority	*1 - Critical*
Design	*2007*
Construct	*2008*
Total Project Cost	*$831,000*
Prior Spending	*$0*
Costs in this Plan Period	*$831,000*

342-06: Construct 24-inch main along Truslow Road and Enon Road to Hulls Chapel Road (8,209 feet)

This project includes design and construction of a 24-inch water main along Truslow Road to Hulls Chapel Road (8,209 feet). The purpose of the project is to convey large quantities of flow from Rocky Pen Run WTP to both the southern and northern zones in the water system. This project significantly increases both the reliability and flexibility of the overall system. The project connects the major transmission mains from the Rocky Pen Run WTP to the 342 Zone by connecting to the existing 16-inch main and the 12-inch main at the Abel Lake Tank, and to the 370N Zone and the northern zones by transferring flows to the 370N Zone Pumping Station near Abel Lake WTP. This project would allow DOU to reduce production or “mothball” the Abel Lake WTP facilities by providing flow from Rocky Pen Run WTP. This flexibility may be important for maintenance of facilities, temporary disruptions in water service (i.e., electrical outages, main breaks, plant shutdowns, etc.), changes in raw water quality, availability of raw water supply, etc.

Priority	*1 – Critical*
Design	*2023*
Construct	*2024*
Total Project Cost	*$1,654,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,654,000*

342-07: Construct 16-inch main from Truslow Road and Beagle Road to Layhill Road at Jefferson Davis Highway (4,992 feet)

This project includes design and construction of a 16-inch water main from Truslow Road and Beagle Road to Layhill Road at Jefferson Davis Highway (4,992 feet). The purpose of the project is to provide a strong connection between the 342 Zone transmission mains from Rocky Pen Run WTP and the 12-inch mains serving future customers in the northern portion of the 342 Zone. The timing for construction of this project is dependent on the timing of water demands in the northern portion of the 342 Zone.

Priority	*2 - Necessary*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Total Project Cost	*$557,000*
Prior Spending	*$0*
Costs in this Plan Period	*$557,000*

342-08: Construct 12-inch main along Layhill Road and Forbes Street from Jefferson Davis Highway to Morton Street (2,711 feet)

This project includes design and construction of a 12-inch water main along Layhill Road and Forbes Street from Jefferson Davis Highway to Morton Street (2,711 feet). The purpose of the project is to provide a second strong connection from the existing 12-inch mains along Jefferson Davis Highway to the 12-inch main along Leeland Road. This connection, in combination with another proposed 12-inch main (342-10), should improve flow and reliability to customers in the vicinity of Grafton Tank. The timing for this connection should be concurrent with the replacement of the Grafton Tank.

Priority	*1 - Critical*
Design	*2004*
Construct	*2005*
Total Project Cost	*$233,000*
Prior Spending	*$0*
Costs in this Plan Period	*$233,000*

342-09: Construct 12-inch main along Forbes Street from Manning Drive to Morton Road (2,647 feet)

This project includes design and construction of a 12-inch water main along Forbes Street from Manning Drive to Morton Road (2,647 feet). The purpose of the project is to serve future customers in the northern portion of the 342 Zone. The timing for construction of this project is dependent on the timing of water demands in the northern portion of the 342 Zone.

Priority	*2 - Necessary*
Design	*2023*
Construct	*2024*
Total Project Cost	*$227,000*
Prior Spending	*$0*
Costs in this Plan Period	*$227,000*

342-10: Construct 12-inch main Morton Road and Elizabeth Drive to Leeland Road and Rive Road (4,144 feet)

This project includes design and construction of a 12-inch water main along Morton Road and Elizabeth Drive to Leeland Road and Rive Road (4,144 feet). The purpose of this project is to connect the 12-inch main on Morton Road and Elizabeth Drive with the 12-inch main on Leeland Road. This connection, in combination with another proposed 12-inch main (342-08), should improve flow and reliability to customers in the vicinity of Grafton Tank. The timing for this connection should approximate the timing for replacement of the Grafton Tank.

Priority	*3 - Prior Appropriation*
Design	*Not Applicable*
Construct	*Not Applicable*
Total Project Cost	*$356,000*
Prior Spending	*$0*
Costs in this Plan Period	*$356,000*

342-12: Construct 12-inch main along Ficklen Road and Conway Road from Sutherland Boulevard to Leeland Road (3,779 feet)

This project includes design and construction of a 12-inch water main along Ficklen Road and Conway Road from Sutherland Boulevard to Leeland Road (3,779 feet). The purpose of the project is to connect the piping served by the 12-inch main along Harrell Road to the 12-inch main along Leeland Road.

Priority	*2 - Necessary*
Design	*2023*
Construct	*2024*
Total Project Cost	*$324,000*
Prior Spending	*$0*
Costs in this Plan Period	*$324,000*

342-13: Construct 12-inch main along Deacon Road from 12-inch main at Dawson Drive to 12-inch near Brooke Road (3,669 feet)

This project includes design and construction of a 12-inch water main along Deacon Road from the 12-inch main at Dawson Drive to the 12-inch main near Brooke Road (3,669 feet). The purpose of the project is to connect the segments of the 12-inch main on Deacon Road and improve flows to the Grafton Tank. The existing 8-inch pipeline experiences high velocity and headloss during high flow conditions. The timing for this connection should approximate the timing for replacement of the Grafton Tank.

Priority	*1 - Critical*
Design	*2004*
Construct	*2005*
Total Project Cost	*$315,000*
Prior Spending	*$0*
Costs in this Plan Period	*$315,000*

342-14: Construct 30-inch main along Greenbank Road, Sanford Drive, Commerce Parkway, Warrenton Road from Rocky Pen Run WTP to Olde Forge Road (22,500 feet)

This project involves design and construction of a 30-inch main along Greenbank Road, Sanford Drive, Commerce Parkway, Warrenton Road from Rocky Pen Run WTP to Olde Forge Road (22,500 feet). The purpose of the project is to convey flows from Rocky Pen Run WTP to the 342, 320 and 370N Zones. Construction of the water main can be delayed by using Rocky Pen Run WTP to serve the 503 Zone and constructing PRVs on the 12-inch mains on Truslow Road and Warrenton Road at the 342 Zone border to offset possible near-term shortfalls in Abel Lake WTP supply to the 342 Zone. A disadvantage of delaying construction of the 30-inch main is the increase in utility conflicts associated with continued development along Warrenton Road.

Priority	*1 - Critical*
Design	*2015*
Construct	*2016*
Total Project Cost	*$4,811,000*
Prior Spending	*$0*
Costs in this Plan Period	*$4,811,000*

342-100: Construct 1.0 MG elevated tank at site of existing Grafton Tank along Deacon Road

Abel Lake WTP currently supplies water to two pressure zones with hydraulic grade lines of 342 feet and 503 feet. Water from the Abel Lake WTP is pumped through a 16-inch water main to the Abel Lake Tank. The Abel Lake Tank is a 4 MG ground level tank with an overflow elevation of 298 feet. Water is pumped from the Abel Lake Tank to the 342 Zone through the Cranes Corner Pumping Station and to the 503 Zone through the Berea Pumping Station. Four elevated storage tanks are located in the 342 Zone:

- Cranes Corner (0.2 MG, 342 ft OF)
- Grafton (0.15 MG, 316 ft OF)
- Ferry Road (1 MG, 320 OF)
- Bandy (0.15 MG, 341 ft OF)

The Cranes Corner and Bandy Tanks are located near the Abel Tank/Cranes Corner Pumping Station while the Grafton and Ferry Road Tanks are distant from the supply. Currently, the Cranes Corner Pumping Station is operated off of the water levels in the Cranes Corner Tank and the pumps in the Cranes Corner Pumping Station cycle on/off due to the small volume of storage in the Cranes Corner Tank. After Rocky Pen Run WTP and the 342 Zone transmission main (30-inch) along Warrenton Road is complete, the 342 Zone will be fed from the vicinity of Warrenton Road and I-95 and the Cranes Corner Pumping Station will be eliminated along with the Abel Tank. The existing Grafton Tank needs to be repainted and was considered for replacement due to its small size and inability to maintain adequate pressures in the area caused by its low overflow elevation (316 feet) and piping constraints in the area. Replacing the Grafton Tank with an elevated tank at a higher overflow elevation will cause the Cranes Corner and Bandy Tanks to typically remain full and not operate properly. Consequently, the small tanks at Cranes Corner (0.2 MG) and Bandy (0.15 MG) will be eliminated following construction of the new Grafton Tank. To improve low pressures in the vicinity of the Grafton Tank, the proposed Grafton Tank will be raised from an overflow elevation of 316 feet to 342 feet. The proposed Grafton Tank will be used to control the pumps at Rocky Pen Run Reservoir WTP that feed the 342 Zone. The Ferry Road Tank, which is in proximity of the proposed Grafton Tank, has an overflow level of 320 feet and will be converted to 320 Zone service. The 320 Zone will be established by closing interconnecting piping in the vicinity of White Oak Road and installing two pressure reducing valves on the transmission mains along Butler Road and White Oak Road.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 342 Zone will be 6.2 mgd. Consequently, the volume of storage needed in the 342 Zone under buildout conditions is roughly 3 MG. In addition to replacement of the Grafton Tank, a second elevated storage tank was proposed at the Cranes Corner Tank site (or alternatively along Leeland Road). Each of the elevated tanks was sized at 1.0 MG with overflow elevations of 342 feet. In addition to the two tanks, a 2 MG elevated tank with a 342 ft overflow is recommended in the vicinity of Abel Lake WTP. This tank would typically provide storage to the 342 Zone in the vicinity of the Abel Lake WTP, and it could feed the larger portion of the 342 Zone if desired (e.g., scheduled maintenance of the future 1.0 MG Grafton Tank). Compared to the existing tank, replacement of the 3,700 feet of 8-inch main with a 12-inch main along Deacon Road should significantly improve the performance of an elevated tank at the Grafton Tank site.

Priority	*1 - Critical*
Design	*2005*
Construct	*2006*
Total Project Cost	*$1,725,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,725,000*

342-101: Construct 2.0 MG elevated tank in vicinity of Mountain View Road

Abel Lake WTP currently supplies water to two pressure zones with hydraulic grade lines of 342 feet and 503 feet. Water from the Abel Lake WTP is pumped through a 16-inch water main to the Abel Lake Tank. The Abel Lake Tank is a 4 MG ground level tank with an overflow elevation of 298 feet. Water is pumped from the Abel Lake Tank to the 342 Zone through the Cranes Corner Pumping Station and to the 503 Zone through the Berea Pumping Station. Four elevated storage tanks are located in the 342 Zone:

- Cranes Corner (0.2 MG, 342 ft OF)
- Grafton (0.15 MG, 316 ft OF)
- Ferry Road (1 MG, 320 OF)
- Bandy (0.15 MG, 341 ft OF)

The Cranes Corner and Bandy Tanks are located near the Abel Tank/Cranes Corner Pumping Station while the Grafton and Ferry Road Tanks are distant from the supply. Currently, the Cranes Corner Pumping Station is operated off of the water levels in the Cranes Corner Tank and the pumps in the Cranes Corner Pumping Station cycle on/off due to the small volume of storage in the Cranes Corner Tank. After Rocky Pen Run WTP and the 342 Zone transmission main (30-inch) along Warrenton Road is complete, the 342 Zone will be fed from the vicinity of Warrenton Road and I-95 and the Cranes Corner Pumping Station will be eliminated along with the Abel Tank. The existing Grafton Tank needs to be repainted and was considered for replacement due to its small size and inability to maintain adequate pressures in the area caused by its low overflow elevation (316 feet) and piping constraints in the area. Replacing the Grafton Tank with an elevated tank at a higher overflow elevation will cause the Cranes Corner and Bandy Tanks to typically remain full and not operate properly. Consequently, the small tanks at Cranes Corner (0.2 MG) and Bandy (0.15 MG) will be eliminated following construction of the new Grafton Tank. To improve low pressures in the vicinity of the Grafton Tank, the proposed Grafton Tank will be raised from an overflow elevation of 316 feet to 342 feet. The proposed Grafton Tank will be used to control the pumps at Rocky Pen Run Reservoir WTP that feed the 342 Zone. The Ferry Road Tank, which is in proximity of the proposed Grafton Tank, has an overflow level of 320 feet and will be converted to 320 Zone service. The 320 Zone will be established by closing interconnecting piping in the vicinity of White Oak Road and installing two pressure reducing valves on the transmission mains along Butler Road and White Oak Road.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 342 Zone will be 6.2 mgd. Consequently, the volume of storage needed in the 342 Zone under buildout conditions is roughly 3 MG. In addition to replacement of the Grafton Tank, a second elevated storage tank was proposed at the Cranes Corner Tank site (or alternatively along Leeland Road). Each of the elevated tanks was sized at 1.0 MG with overflow elevations of 342 feet. In addition to the two tanks, a 2 MG elevated tank with a 342 ft overflow is recommended in the vicinity of Abel Lake WTP. This tank would typically provide storage to the 342 Zone in the vicinity of the Abel Lake WTP, and it could feed the larger portion of the 342 Zone if desired (e.g., scheduled maintenance of the future 1.0 MG Grafton Tank). In addition, this tank would provide suction storage for the pumping station serving the 370N Zone. The tank would be refilled from either Rocky Pen Run WTP or Abel Lake WTP or both. The pumps at the Abel Lake WTP would need to be modified to pump from the Abel Lake Tank at overflow elevation 298 feet to the proposed elevated tank at 342 feet. The configuration of the improvements proposed in the vicinity of Abel Lake WTP are shown in Appendix C of this technical memorandum.

Priority	*1 - Critical*
Design	*2018*
Construct	*2019*
Total Project Cost	*$3,450,000*
Prior Spending	*$0*
Costs in this Plan Period	*$3,450,000*

342-102: Construct 1.0 MG elevated tank at site of existing Cranes Corner Tank along Jefferson Davis Highway

Abel Lake WTP currently supplies water to two pressure zones with hydraulic grade lines of 342 feet and 503 feet. Water from the Abel Lake WTP is pumped through a 16-inch water main to the Abel Lake Tank. The Abel Lake Tank is a 4 MG ground level tank with an overflow elevation of 298 feet. Water is pumped from the Abel Lake Tank to the 342 Zone through the Cranes Corner Pumping Station and to the 503 Zone through the Berea Pumping Station. Four elevated storage tanks are located in the 342 Zone:

- Cranes Corner (0.2 MG, 342 ft OF)
- Grafton (0.15 MG, 316 ft OF)
- Ferry Road (1 MG, 320 OF)
- Bandy (0.15 MG, 341 ft OF)

The Cranes Corner and Bandy Tanks are located near the Abel Tank/Cranes Corner Pumping Station while the Grafton and Ferry Road Tanks are distant from the supply. Currently, the Cranes Corner Pumping Station is operated off of the water levels in the Cranes Corner Tank and the pumps in the Cranes Corner Pumping Station cycle on/off due to the small volume of storage in the Cranes Corner Tank. After Rocky Pen Run WTP and the 342 Zone transmission main (30-inch) along Warrenton Road is complete, the 342 Zone will be fed from the vicinity of Warrenton Road and I-95 and the Cranes Corner Pumping Station will be eliminated along with the Abel Tank. The existing Grafton Tank needs to be repainted and was considered for replacement due to its small size and inability to maintain adequate pressures in the area caused by its low overflow elevation (316 feet) and piping constraints in the area. Replacing the Grafton Tank with an elevated tank at a higher overflow elevation will cause the Cranes Corner and Bandy Tanks to typically remain full and not operate properly. Consequently, the small tanks at Cranes Corner (0.2 MG) and Bandy (0.15 MG) will be eliminated following construction of the new Grafton Tank. To improve low pressures in the vicinity of the Grafton Tank, the proposed Grafton Tank will be raised from an overflow elevation of 316 feet to 342 feet. The proposed Grafton Tank will be used to control the pumps at Rocky Pen Run Reservoir WTP that feed the 342 Zone. The Ferry Road Tank, which is in proximity of the proposed Grafton Tank, has an overflow level of 320 feet and will be converted to 320 Zone service. The 320 Zone will be established by closing interconnecting piping in the vicinity of White Oak Road and installing two pressure reducing valves on the transmission main along Butler Road and White Oak Road.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 342 Zone will be 6.2 mgd. Consequently, the volume of storage needed in the 342 Zone under buildout conditions is roughly 3 MG. In addition to replacement of the Grafton Tank, a second elevated storage tank was proposed at the Cranes Corner Tank site (or alternatively along Leeland Road). Each of the elevated tanks was sized at 1.0 MG with overflow elevations of 342 feet. In addition to the two tanks, a 2 MG elevated tank with a 342 ft overflow is recommended in the vicinity of Abel Lake WTP.

The new 1.0 MG elevated tank at the Cranes Corner Tank site will serve the area along the Jefferson Davis Highway corridor using the existing 12-inch water main.

Priority	*2 - Necessary*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Total Project Cost	*$1,725,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,725,000*

6.6. 370N Zone Improvements

370N-01: Construct 24-inch main along Centreport Parkway from Abel Lake WTP to Ramoth Church Road (15,402 feet)

This project includes design and construction of a 24-inch water main along Centreport Parkway from Abel Lake WTP to Ramoth Church Road (15,402 feet). The purpose of the project is to convey large quantities of flow from Abel Lake WTP to the customers in the Centreport Industrial Park, the future 370N Zone, and zones in the northern portion of the County.

Priority	*3 – Prior Appropriation*
Design	*Not Applicable*
Construct	*Not Applicable*
Total Project Cost	*$2,495,000*
Prior Spending	*$0*
Costs in this Plan Period	*$2,495,000*

370N-02: Construct 12-inch main along Ramoth Church Road and American Legion Road from 24-inch at Ramoth Church Road to State Shop Road (3,566 feet)

This project includes design and construction of a 12-inch water main along Ramoth Church Road and American Legion Road from the 24-inch main at Ramoth Church Road to State Shop Road (3,566 feet). The purpose of the project is to create a strong connection under I-95 from the proposed transmission main on Centreport Parkway to the existing 12-inch main on Jefferson Davis Highway.

Priority	*1 - Critical*
Design	*2020*
Construct	*2021*
Total Project Cost	*$526,000*
Prior Spending	*$0*
Costs in this Plan Period	*$526,000*

370N-03: Construct 18-inch main from Ramoth Church Road to Courthouse Road (9,137 feet)

This project includes design and construction of an 18-inch water main from the 24-inch along Ramoth Church Road to Courthouse Road (9,137 feet). The purpose of the project is to convey flow north from Abel Lake and Rocky Pen Run WTPs to the northern zones and to the 433 Zone through the proposed pumping station along Courthouse Road. Currently, a single 12-inch main along Jefferson Davis Highway conveys flow through the future 370N Zone. The proposed 18-inch transmission main would be a significant component in DOU's ability to transfer large quantities of flow to the northern zones from Abel Lake and Rocky Pen Run WTPs and to the southern zones from Smith Lake WTP; thereby providing a high level of overall system reliability. As the area of the 370N Zone west of I-95 develops, DOU could construct a network of 16-inch, 12-inch and 8-inch mains to provide the transmission capacity needed to achieve the level of reliability associated with the proposed 18-inch main. Alternatively, the 12-inch main along Jefferson Davis Highway from Ramoth Church Road to Courthouse Road could be replaced with a larger main to increase transmission capacity through the 370N Zone.

Under buildout conditions, demands for the 433 Zone (7.3 mgd) and transfers to the 472 Zone (2.7 mgd) will be satisfied by the Moncure PS, the 433 Zone PS along Courthouse Road, and the Mountain View PS. As an alternative to construction of the 433 Zone Pumping Station along Courthouse Road and possibly the 18-inch main from Ramoth Church Road to Courthouse Road (370N-02), DOU could consider expanding the 370N Zone PS to include 433 Zone pumps. The 433 Zone pumps could feed the 433 Zone and the future 472 Zone PS on Lightfoot Road near Mountain View Road (472-200) through short segment of 12-inch main connecting the 370N Zone PS to the existing 12-inch main along

Mountain View Road. This configuration is not presented as the recommended approach for several reasons:

- The 12-inch main along Mountain View Road is distant from the demand center which is further north in the 433 Zone. Supplying the flow near the demand center improves system reliability.
- The piping network through the 433 Zone from the 12-inch main along Mountain View Road is weak compared with the network connected to the 12-inch main along Courthouse Road.
- The recommended projects create a strong transmission artery between the three treatment plants and along the I-95/Jefferson Davis Highway corridor.

The configuration of the improvements and flow balance schematics proposed in the vicinity of Abel Lake WTP are shown in Appendix A and C of this technical memorandum, respectively.

Priority	*2 - Necessary*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Total Project Cost	*$1,140,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,140,000*

370N-04: Construct 12-inch main along Courthouse Road from west of I-95 east to Jefferson Davis Highway (5,156 feet)

This project includes design and construction of a 12-inch water main along Courthouse Road from west of I-95 east to Jefferson Davis Highway (5,156 feet). The purpose of the project is to create a strong connection under I-95 from the existing 18-inch main serving Embrey Mill to the existing 12-inch main on Jefferson Davis Highway. The timing for construction of this project will depend on the timing for expansion of the 370N Zone. Replacement of the Courthouse Tank at an overflow elevation of 370 feet (370N-100) would allow the 370N Zone to be expanded from the Embrey Mill area to the Courthouse Tank area. This connection would provide the flow needed for satisfactory drawdown and refill of the Embrey Mill (370 ft overflow) and Courthouse Tanks (370 ft overflow). If DOU replaces (or plans to replace) the 12-inch main along Jefferson Davis Highway in the 370N Zone with a larger main in lieu of the 18-inch main west of I-95 (370N-03), DOU should consider constructing a main larger than 12 inches for the connection along Courthouse Road. Creating a stronger connection would improve operation of the 370N Zone tanks at Embrey Mill and Courthouse and provide additional flow to proposed 433 Zone pumping station along Courthouse Road (433-200).

Priority	*1 - Critical*
Design	*2020*
Construct	*2021*
Total Project Cost	*$670,000*
Prior Spending	*$0*
Costs in this Plan Period	*$670,000*

370N-05: Construct 16-inch main along Courthouse Road from west of I-95 west to 433 Zone pumping station near Snowbird Lane (3,496 feet)

This project includes design and construction of a 16-inch water main along Courthouse Road from west of I-95 west to 433 Zone pumping station near Snowbird Lane (3,496 feet). The purpose of the project is to provide flow from the existing 18-inch water main in Embrey Mill and the proposed 18-inch water main (370N-03) to the proposed 433 Zone Pumping Station. This pumping station will provide a second source of supply to the 433 Zone and utilize the transmission system in the southern portion of the 433 Zone to deliver flow to the customers in the southern portion of the 433 Zone and to the 472 Zone. The timing for construction of this main will be concurrent with the 433 Zone Pumping Station (433-200).

Priority	*1 - Critical*
Design	*2020*
Construct	*2021*
Total Project Cost	*$396,000*
Prior Spending	*$0*
Costs in this Plan Period	*$396,000*

370N-100: Construct 1.0 MG elevated tank along Jefferson Davis Highway near Clarke Hill Road

In the near-term, the 370N Zone will be limited to Embrey Mill area which will be served by the Embrey Mill Pumping Station (370N-200) and a 0.5 MG tank in Embrey Mill (370N-101). The proposed Embrey Mill Pumping Station will be sized for 2.0 mgd. As demands increase through the planning period and more flow from Smith Lake WTP is delivered south, flows will be transferred from the 370N Zone south to the Courthouse Tank area through a PRV. In lieu of installing a PRV, the Courthouse Tank could be replaced and the 370N Zone could be expanded south of Embrey Mill to include the area in the vicinity of the Courthouse Tank. The southern portion of the 370N Zone will be established with the construction of the 342 Zone Tank (342-101) and 370N Zone Pumping Station (370N-201) near Abel Lake WTP. After the 370N Zone is established, the Embrey Mill PS will serve as an emergency backup for delivering flow from Smith Lake WTP to the southern pressure zones.

The configuration of the improvements and flow balance schematics proposed in the vicinity of Abel Lake WTP are shown in Appendix A and C of this technical memorandum, respectively.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 370N Zone will be 4.0 mgd. Consequently, the volume of storage needed in the 370N Zone under buildout conditions is roughly 2.0 MG. This project includes construction of 1.0 MG of elevated storage at an overflow elevation of 370 feet. A second 0.5 MG elevated tank is proposed for Embrey Mill and the remaining 0.5 MG will be satisfied from the 342 Zone Tank and the 370N Zone PS.

Priority	*1 - Critical*
Design	*2021*
Construct	*2022*
Total Project Cost	*$1,725,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,725,000*

370N-101: Construct 0.5 MG elevated tank in Embrey Mill north of Courthouse Road

In the near-term, the 370N Zone will be limited to Embrey Mill area which will be served by the Embrey Mill Pumping Station (370N-200) and a 0.5 MG tank in Embrey Mill (370N-101). The proposed Embrey Mill Pumping Station will be sized for 2.0 mgd. As demands increase through the planning period and more flow from Smith Lake WTP is delivered south, flows will be transferred from the 370N Zone south to the Courthouse Tank area through a PRV. In lieu of installing a PRV, the Courthouse Tank could be replaced and the 370N Zone could be expanded south of Embrey Mill to include the area in the vicinity of the Courthouse Tank. The southern portion of the 370N Zone will be established with the construction of the 342 Zone Tank (342-101) and 370N Zone Pumping Station (370N-201) near Abel Lake WTP. After the 370N Zone is established, the Embrey Mill PS will serve as an emergency backup for delivering flow from Smith Lake WTP to the southern pressure zones.

The configuration of the improvements and flow balance schematics are shown in Appendix A and C of this technical memorandum, respectively.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 370N Zone will be 4.0 mgd. Consequently, the volume of storage needed in the 370N Zone under buildout conditions is roughly 2.0 MG. This project includes construction of 1.0 MG of elevated storage at an overflow elevation of 370 feet. A second 0.5 MG elevated tank is proposed for Embrey Mill and the remaining 0.5 MG will be satisfied from the 342 Zone Tank and the 370N Zone PS.

Modeling runs indicated that constructing a 0.5 MG storage tank in Embrey Mill significantly improved fire flows in Embrey Mill compared with providing this storage for Embrey Mill at the Courthouse Tank site. In addition, a tank in Embrey Mill along with the emergency backup service from the Embrey Mill PS will significantly improve reliability in the northern portion of the 370N Zone which will be distant from the proposed pumping station for the 370N Zone near Abel Lake WTP.

Priority	*4 - Developer*
Design	*Not Applicable*
Construct	*Not Applicable*
Total Project Cost	*$863,000*
Prior Spending	*$0*
Costs in this Plan Period	*$863,000*

370N-200: Construct 2.0 mgd Embrey Mill Pumping Station near Wallace Lane

In the near-term, the 370N Zone will be limited to Embrey Mill area which will be served by the Embrey Mill Pumping Station (370N-200) and a 0.5 MG tank in Embrey Mill (370N-101). The proposed Embrey Mill Pumping Station will be sized for 2.0 mgd. As demands increase through the planning period and more flow from Smith Lake WTP is delivered south, flows will be transferred from the 370N Zone south to the Courthouse Tank area through a PRV. In lieu of installing a PRV, the existing Courthouse Tank (310 ft OF) could be replaced with a tank at 370 ft overflow and the 370N Zone could be expanded south of Embrey Mill to include the area in the vicinity of the Courthouse Tank. The southern portion of the 370N Zone will be established with the construction of the 342 Zone Tank (342-101) and 370N Zone Pumping Station (370N-201) near Abel Lake WTP. After the 370N Zone is established, the Embrey Mill PS will serve as an emergency backup for delivering flow from Smith Lake WTP to the southern pressure zones.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 370N Zone will be 4.0 mgd. Consequently, the volume of storage needed in the 370N Zone under buildout conditions is roughly 2.0 MG. This project includes construction of 1.0 MG of elevated storage at an overflow elevation of 370 feet. A second 0.5 MG elevated tank is proposed for Embrey Mill and the remaining 0.5 MG will be satisfied from the 342 Zone Tank and the 370N Zone PS.

The boundary for the 370N Zone was established based on ground elevations obtained from County GIS data along with maximum and minimum pressure requirements. Modeling runs indicated that constructing a 0.5 MG storage tank in Embrey Mill significantly improved fire flows in Embrey Mill compared with providing this storage for Embrey Mill at the Courthouse Tank site. In addition, a tank in Embrey Mill along with the emergency backup service from the Embrey Mill PS will significantly improve reliability in the northern portion of the 370N Zone which will be distant from the proposed pumping station for the 370N Zone near Abel Lake WTP.

Constructing a 2.0 mgd pumping station on the 370N/310 Zone boundary is recommended for the near-term for the following reasons:

- Pumping is from higher ground elevations in the northern portion of the 370N Zone to the lower areas in the zone.
- The pumping station is proposed in a location near the proposed demand center (i.e., Embrey Mill).
- Prior to bringing Rocky Pen Run WTP on-line, this pumping station can be used to move water from Smith Lake WTP to the southern zones in the water system thereby reducing the production requirements for Abel Lake WTP.
- After Rocky Pen Run WTP is on-line, the Embrey Mill PS is a key redundancy feature allowing flow from Smith Lake WTP to be conveyed to the southern zones in the County during emergencies.

Further expansion of the Embrey Mill PS will be dependent on the expansion at Moncure PS and the projected near-term demands. To fully utilize the available production capacity of Smith Lake WTP (14 mgd), roughly 10.5 mgd will need to be transferred through the Embrey Mill and Moncure Pumping Stations (i.e., 14 mgd production from Smith Lake WTP – 3.5 mgd near-term maximum day demand for 310 Zone).

The configuration of the improvements and flow balance schematics are shown in Appendix A and C of this technical memorandum, respectively.

Priority	*1 - Critical*
Design	*2007*
Construct	*2008*
Total Project Cost	*$400,000*
Prior Spending	*$0*
Costs in this Plan Period	*$400,000*

370N-201: Construct 10 mgd Abel Lake Pumping Station in vicinity of Mountain View Road

In the near-term, the 370N Zone will be limited to Embrey Mill area which will be served by the Embrey Mill Pumping Station (370N-200) and a 0.5 MG tank in Embrey Mill (370N-101). The proposed Embrey Mill Pumping Station will be sized for 2.0 mgd. As demands increase through the planning period and more flow from Smith Lake WTP is delivered south, flows will be transferred from the 370N Zone south to the Courthouse Tank area through a PRV. In lieu of installing a PRV, the existing Courthouse Tank (310 ft OF) could be replaced with a tank at 370 ft overflow and the 370N Zone could be expanded south of Embrey Mill to include the area in the vicinity of the Courthouse Tank. The southern portion of the 370N Zone will be established with the construction of the 342 Zone Tank (342-101) and 370N Zone Pumping Station (370N-201) near Abel Lake WTP. After the 370N Zone is established, the Embrey Mill PS will serve as an emergency backup for delivering flow from Smith Lake WTP to the southern pressure zones.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 370N Zone will be 4.0 mgd. Consequently, the volume of storage needed in the 370N Zone under buildout conditions is roughly 2.0 MG. This project includes construction of 1.0 MG of elevated storage at an overflow elevation of 370 feet. A second 0.5 MG elevated tank is proposed for Embrey Mill and the remaining 0.5 MG will be satisfied from the 342 Zone Tank and the 370N Zone PS.

The boundary for the 370N Zone was established based on ground elevations obtained from County GIS data along with maximum and minimum pressure requirements. Modeling runs indicated that constructing a 0.5 MG storage tank in Embrey Mill significantly improved fire flows in Embrey Mill compared with providing this storage for Embrey Mill at the Courthouse Tank site. In addition, a tank in Embrey Mill along with the emergency backup service from the Embrey Mill PS will significantly improve reliability in the northern portion of the 370N Zone which will be distant from the proposed pumping station for the 370N Zone near Abel Lake WTP.

Constructing a 2.0 mgd pumping station on the 370N/310 Zone boundary is recommended for the near-term for the following reasons:

- Pumping is from higher ground elevations in the northern portion of the 370N Zone to the lower areas in the zone.
- The pumping station is proposed in a location near the proposed demand center (i.e., Embrey Mill).
- Prior to bringing Rocky Pen Run WTP on-line, this pumping station can be used to move water from Smith Lake WTP to the southern zones in the water system thereby reducing the production requirements for Abel Lake WTP.
- After Rocky Pen Run WTP is on-line, the Embrey Mill PS is a key redundancy feature allowing flow from Smith Lake WTP to be conveyed to the southern zones in the County during emergencies.

Further expansion of the Embrey Mill PS will be dependent on the expansion at Moncure PS and the projected near-term demands. To convey the available production capacity of Smith Lake WTP (14 mgd), roughly 10.5 mgd will need to be transferred through the Embrey Mill and Moncure Pumping Stations (i.e., 14 mgd production from Smith Lake WTP – 3.5 mgd near-term maximum day demand for 310 Zone).

The proposed 370N Zone Pumping Station will be approximately 10 mgd with a pumping head of 60-70 feet. To satisfy the remaining 0.5 MG storage deficit in the 370N Zone, it may be necessary to size the pumping station for a demand condition greater than the maximum day demand of 10 mgd. Suction for the pumping units will be from the proposed 342 Zone Tank (2.0 MG, 342 ft OF). The pumping station will be capable of meeting the maximum day demand of the 370N Zone at buildout (6 mgd) which will be supplied by Abel Lake WTP plus a transfer of 4 mgd from Rocky Pen Run WTP to the 433 Zone.

Under buildout conditions, demands for the 433 Zone (7.3 mgd) and transfers to the 472 Zone (2.7 mgd) will be satisfied by the Moncure PS, 433 Zone PS along Courthouse Road, and the Mountain View PS. As an alternative to construction of the 433 Zone Pumping Station along Courthouse Road and possibly the 18-inch main from Ramoth Church Road to Courthouse Road (370N-03), DOU could consider expanding the 370N Zone PS to include 433 Zone pumps. The 433 Zone pumps could feed the 433 Zone and the future 472 Zone PS on Lightfoot Road near Mountain View Road (472-200) through short segment of 12-inch main connecting the 370N Zone PS to the existing 12-inch main along Mountain View Road. This configuration is not presented as the recommended approach for several reasons:

- The 12-inch main along Mountain View Road is distant from the demand center which is further north in the 433 Zone. Supplying the flow near the demand center improves system reliability.
- The piping network through the 433 Zone from the 12-inch main along Mountain View Road is weak compared with the network connected to the 12-inch main along Courthouse Road.

- The recommended projects create a strong transmission artery between the three treatment plants and along the I-95/Jefferson Davis Highway corridor.

The configuration of the improvements and flow balance schematics are shown in Appendix A and C of this technical memorandum, respectively.

Priority	*1 - Critical*
Design	*2019*
Construct	*2020*
Total Project Cost	*$2,430,000*
Prior Spending	*$0*
Costs in this Plan Period	*$2,430,000*

6.7. 370S Zone Improvements

<u>370S-01: Construct 16-inch main along Virginia Parkway from PRV north of Sanford Road to end of Virginia Parkway (5,227 feet)</u>

Two pressure reducing valves (PRVs) are proposed on transmission mains along the southern border of the 480 Zone to provide flow from the 480 Zone to the 370S Zone. In the near-term, flow to the 370S Zone will be provided by the Abel Lake WTP from the 503 Zone. After Rocky Pen Run WTP is on-line, flow to the 370S Zone will be provided by Rocky Pen Run WTPs through the 480 Zone. Alternatively, one option that was considered in lieu of the PRVs was construction of a pumping station on the 30-inch transmission main serving the 342 Zone. Due to the small quantity of flow projected for the 370S Zone at buildout (average day demand of 1.1 mgd), it was concluded that operation and maintenance of a separate pumping station was not cost-effective. A second alternative that was considered is installation of 370S Zone pumps in the pumping station at Rocky Pen Run WTP and construction of a water main along Sanford Drive from the pumping station to the 16-inch main along Greenbank Road. While this is a feasible approach, supplying the 370S Zone through the two connections to the existing 503 Zone and future 480 Zone at the PRVs provides a higher level of reliability and is more cost-effective.

This project involves design and construction of a 16-inch main along Virginia Parkway from PRV north of Sanford Road to end of Virginia Parkway (5,227 feet). The sizing for the 16-inch water main was based on achieving a 2,500 gpm fire flow at the end of the 16-inch main.

Priority	*4 - Developer*
Design	*Not Applicable*
Construct	*Not Applicable*
Total Project Cost	*$616,000*
Prior Spending	*$0*
Costs in this Plan Period	*$616,000*

<u>370S-300: Construct pressure reducing valve between 503/370S Zone along Virginia Parkway between Commerce Parkway and Sanford Drive</u>

Two pressure reducing valves (PRVs) are proposed on transmission mains along the southern border of the 480 Zone to provide flow from the 480 Zone to the 370S Zone. In the near-term, flow to the 370S Zone will be provided by the Abel Lake WTP from the 503 Zone. After Rocky Pen Run WTP is on-line, flow to the 370S Zone will be provided by Rocky Pen Run WTPs through the 480 Zone. Alternatively, one option that was considered in lieu of the PRVs was a pumping station on the 30-inch transmission main serving the 342 Zone. Due to the small quantity of flow projected for the 370S Zone at buildout (average day demand of 1.1 mgd), it was concluded that operation and maintenance of a separate pumping station was not cost-effective. A second alternative that was considered is installation of 370S Zone pumps in the pumping station at Rocky Pen Run WTP and construction of a water main along Sanford Drive from the pumping station to the 16-inch main along Greenbank Road. While this is a feasible approach, supplying the 370S Zone through the two connections to the existing 503 Zone and future 480 Zone at the PRVs provides a higher level of reliability ad is more cost-effective.

Priority	*4 - Developer*
Design	*Not Applicable*
Construct	*Not Applicable*
Total Project Cost	*$65,000*
Prior Spending	*$0*
Costs in this Plan Period	*$65,000*

370S-301: Construct pressure reducing valve between 503/370S Zone along Warrenton Road near Sanford Drive

Two pressure reducing valves (PRVs) are proposed on transmission mains along the southern border of the 480 Zone to provide flow from the 480 Zone to the 370S Zone. In the near-term, flow to the 370S Zone will be provided by the Abel Lake WTP from the 503 Zone. After Rocky Pen Run WTP is on-line, flow to the 370S Zone will be provided by Rocky Pen Run WTPs through the 480 Zone. Alternatively, one option that was considered in lieu of the PRVs was a pumping station on the 30-inch transmission main serving the 342 Zone. Due to the small quantity of flow projected for the 370S Zone at buildout (average day demand of 1.1 mgd), it was concluded that operation and maintenance of a separate pumping station was not cost-effective. A second alternative that was considered is installation of 370S Zone pumps in the pumping station at Rocky Pen Run WTP and construction of a water main along Sanford Drive from the pumping station to the 16-inch main along Greenbank Road. While this is a feasible approach, supplying the 370S Zone through the two connections to the existing 503 Zone and future 480 Zone at the PRVs provides a higher level of reliability and is more cost-effective.

Priority	*4 - Developer*
Design	*Not Applicable*
Construct	*Not Applicable*
Total Project Cost	*$65,000*
Prior Spending	*$0*
Costs in this Plan Period	*$65,000*

6.8. 433 Zone Improvements

433-01: Construct 12-inch main along Old Mineral Road from Northhampton Boulevard to Highpointe Boulevard (1,558 feet)

This project includes design and construction of a 12-inch water main along Old Mineral Road from Northhampton Boulevard to Highpointe Boulevard (1,558 feet). The purpose of the project is to connect the proposed 12-inch main along Cobblers Court and Brafferton Boulevard and the existing 12-inch main along Highpointe Boulevard to the 12-inch main along Northhampton Boulevard. This project, in combination with the 12-inch main along Cobblers Court and Brafferton Boulevard (433-02), significantly improves the ability to convey flow south and west of the Moncure PS across the 433 Zone. The timing for this project is beyond 2025 which illustrates that the project will be dependent on the magnitude and timing of demands in the 433 Zone and the ability of the pumping stations feeding the southern portion of the 433 Zone to meet demands.

Priority	*2 - Necessary*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Total Project Cost	*$160,000*
Prior Spending	*$0*
Costs in this Plan Period	*$160,000*

433-02: Construct 12-inch main along Cobblers Court and Brafferton Boulevard from Highpointe Boulevard to Garrisonville Road (4,482 feet)

This project includes design and construction of a 12-inch water main along Cobblers Court and Brafferton Boulevard from Highpointe Boulevard to Garrisonville Road (4,482 feet). The purpose of the project is to connect the proposed 16-inch main along Garrisonville Road near the Moncure PS to the 12-inch main along Highpointe Boulevard. This project, in combination with the 12-inch main along Old Mineral Road (433-01) significantly improves the ability to convey flow south and west of the Moncure PS across the 433 Zone. The timing for this project is beyond 2025 which illustrates that the project will be dependent on the magnitude and timing of demands in the 433 Zone and the ability of the pumping stations feeding the southern portion of the 433 Zone to meet demands.

Priority	*2 - Necessary*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Total Project Cost	*$452,000*
Prior Spending	*$0*
Costs in this Plan Period	*$452,000*

433-03: Construct 16-inch main along Garrisonville Road from Moncure Pumping Station to Onville Road (3,191 feet)

This project includes design and construction of a 16-inch water main along Garrisonville Road from Moncure Pumping Station to Onville Road (3,191 feet). The purpose of the project is to convey flows to customers along the Garrisonville Road corridor, particularly the Quantico Marine Corps Base which is fed by the 12-inch main along Onville Road.

Priority	*3 – Prior Appropriation*
Design	*Not Applicable*
Construct	*Not Applicable*
Total Project Cost	*$362,000*
Prior Spending	*$0*

Costs in this Plan Period *$362,000*

433-04: Construct 12-inch main along Danielle Way from 12-inch main to pipes serving future development (6,000 feet)

This project includes design and construction of 12-inch water mains along Embrey Mill Road and Danielle Way from existing 12-inch main to pipes serving the future development (6,000 feet). The purpose of these projects is to connect the existing 12-inch mains along Courthouse Road and Winding Creek Road to the piping network for the future development.

Priority	*4 - Developer*
Design	*Not Applicable*
Construct	*Not Applicable*
Total Project Cost	*$544,000*
Prior Spending	*$0*
Costs in this Plan Period	*$544,000*

433-05: Construct 16-inch main along Courthouse Road from pumping station at 433/370N Zone boundary to Danielle Way (5,147 feet)

This project includes design and construction of a 16-inch main along Courthouse Road from pumping station at 433/370N Zone boundary to Danielle Way (5,147 feet). The purpose of the project is to provide flow from the pumping station to the 12-inch mains on Courthouse Road, Danielle Way and Ramoth Church Road. This pumping station will provide a second source of supply to the 433 Zone and utilize the transmission system in the southern portion of the 433 Zone to deliver flow to the customers in the southern portion of the 433 Zone and to the 472 Zone. The timing for construction of this main will be concurrent with the 433 Zone Pumping Station (433-200).

Priority	*1 - Critical*
Design	*2020*
Construct	*2021*
Total Project Cost	*$584,000*
Prior Spending	*$0*
Costs in this Plan Period	*$584,000*

433-06: Construct 12-inch main from Moncure Pumping Station to 10-inch main south of the pumping station (327 feet)

This project includes design and construction of a 12-inch water main from Moncure Pumping Station to 10-inch main south of the pumping station (327 feet). The purpose of the project is to strengthen the connection to the water system south of Garrisonville Road in the immediate vicinity of the Moncure PS.

Priority	*3 – Prior Appropriation*
Design	*Not Applicable*
Construct	*Not Applicable*
Total Project Cost	*$30,000*
Prior Spending	*$0*
Costs in this Plan Period	*$30,000*

433-07: Construct 12-inch main along Kellogg Mill Road through intersection with Ramoth Church Road (330 feet)

This project includes design and construction of a 12-inch water main along Kellogg Mill Road through intersection with Ramoth Church Road (330 feet). The purpose of the project is to connect the 12-inch mains along Kellogg Mill Road to improve flows through the existing mains in this intersection.

Priority	*2 - Necessary*
Design	*2014*
Construct	*2015*
Total Project Cost	*$63,000*
Prior Spending	*$0*
Costs in this Plan Period	*$63,000*

433-100: Replace Shelton Shop Standpipe with 1.0 MG Elevated Tank

Smith Lake WTP currently supplies water to three pressure zones with hydraulic grade lines of 310, 433 and 472 feet. Water from the Smith Lake WTP is pumped to the Moncure PS on the western border of the 310 Zone which pumps flow to the 433 Zone. Flow from the 433 Zone is boosted to the 472 Zone through the Vista Woods PS which is located on the western border of the 433 Zone along Shelton Shop Road. The 472 Zone has one elevated tank along Mountain View Road in the vicinity of Spy Glass Lane (0.5 MG Vista Woods Tank at overflow elevation 472 feet).

Under buildout conditions, demands for the 433 Zone (7.3 mgd) and transfers to the 472 Zone (2.7 mgd) will be satisfied by the Moncure PS, 433 Zone PS along Courthouse Road, and the Mountain View PS. As an alternative to construction of the 433 Zone Pumping Station along Courthouse Road and possibly the 18-inch main from Ramoth Church Road to Courthouse Road (370N-03), DOU could consider expanding the 370N Zone PS to include 433 Zone pumps. The 433 Zone pumps could feed the 433 Zone and the future 472 Zone PS on Lightfoot Road near Mountain View Road (472-200) through short segment of 12-inch main connecting the 370N Zone PS to the existing 12-inch main along Mountain View Road. This configuration is not presented as the recommended approach for several reasons:

- The 12-inch main along Mountain View Road is distant from the demand center which is further north in the 433 Zone. Supplying the flow near the demand center improves system reliability.
- The piping network through the 433 Zone from the 12-inch main along Mountain View Road is weak compared with the network connected to the 12-inch main along Courthouse Road.
- The recommended projects create a strong transmission artery between the three treatment plants and along the I-95/Jefferson Davis Highway corridor.

Currently, the Moncure PS is operated off of the water levels in the Shelton Shop Tank. In the near-term future, the Moncure PS would be upgraded and expanded to 8.5 mgd and a second 1.4 mgd pumping station is proposed on the 12-inch main on Lightfoot Drive at the intersection of Mountain View Road at the 472/433 Zone border. The 3.0 mgd of combined pumping capacity from the two stations satisfies the projected buildout demand of 2.7 mgd in the 472 Zone. Due to the future pumping and piping configuration through the 433 Zone, the pumping stations serving the 472 Zone would primarily be served by separate supply sources:

- Vista Woods PS would essentially be supplied from Smith Lake WTP through the Moncure PS and water mains along Garrisonville Road.
- Lightfoot Drive PS would be fed from the Abel Lake and Rocky Pen Run WTPs through the pumping station along Courthouse Road at the 370N/433 Zone border.

Expansion of the Moncure PS will be dependent on the expansion at Embrey Mill PS and the projected near-term demands. To fully utilize the available production capacity of Smith Lake WTP (14 mgd), roughly 10.5 mgd will need to be transferred through the Embrey Mill and Moncure Pumping Stations (i.e., 14 mgd production from Smith Lake WTP – 3.5 mgd near-term maximum day demand for 310 Zone). Under the option presented in this Master Plan, the Embrey Mill PS was sized for 2 mgd and the Moncure PS will need to be capable of pumping 8.5 mgd. The configuration of the improvements and flow balance schematics are shown in Appendix A and C of this technical memorandum, respectively.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 433 Zone will be 4.9 mgd. Consequently, the volume of storage needed in the 472 Zone under buildout conditions is roughly 2.5 MG. At buildout, the Amyclae Tank will provide 1.5 MG. Due to the lack of useable storage in the Shelton Shop Standpipe, DOU should consider replacing the standpipe with a 1.0 MG elevated storage tank to satisfy the remaining 1.0 MG deficit. The Shelton Shop elevated tank will provide suction storage for the Vista Woods PS.

Priority	*2 - Necessary*
Design	*2021*
Construct	*2022*
Total Project Cost	*$1,725,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,725,000*

433-200: Construct 2.8 mgd 433 Zone Pumping Station along Courthouse Road near Snowbird Lane

Smith Lake WTP currently supplies water to three pressure zones with hydraulic grade lines of 310, 433 and 472 feet. Water from the Smith Lake WTP is pumped to the Moncure PS on the western border of the 310 Zone which pumps flow to the 433 Zone. Flow from the 433 Zone is boosted to the 472 Zone through the Vista Woods PS which is located on the western border of the 433 Zone along Shelton Shop Road. The 472 Zone has one elevated tank along Mountain View Road in the vicinity of Spy Glass Lane (0.5 MG Vista Woods Tank at overflow elevation 472 feet).

Under buildout conditions, demands for the 433 Zone (7.3 mgd) and transfers to the 472 Zone (2.7 mgd) will be satisfied by the Moncure PS, 433 Zone PS along Courthouse Road, and the Mountain View PS. As an alternative to construction of the 433 Zone Pumping Station along Courthouse Road (2.8 mgd) and possibly the 18-inch main from Ramoth Church Road to Courthouse Road (370N-03), DOU could consider expanding the 370N Zone PS to include 433 Zone pumps. The 433 Zone pumps could feed the 433 Zone and the future 472 Zone PS on Lightfoot Road near Mountain View Road (472-200) through short segment of 12-inch main connecting the 370N Zone PS to the existing 12-inch main along Mountain View Road. This configuration is not presented as the recommended approach for several reasons:

- The 12-inch main along Mountain View Road is distant from the demand center which is further north in the 433 Zone. Supplying the flow near the demand center improves system reliability.
- The piping network through the 433 Zone from the 12-inch main along Mountain View Road is weak compared with the network connected to the 12-inch main along Courthouse Road.
- The recommended projects create a strong transmission artery between the three treatment plants and along the I-95/Jefferson Davis Highway corridor.

Currently, the Moncure PS is operated off of the water levels in the Shelton Shop Tank. In the near-term future, the Moncure PS would be upgraded and expanded to 8.5 mgd and a second 1.4 mgd pumping station is proposed on the 12-inch main on Lightfoot Drive at the intersection of Mountain View Road at the 472/433 Zone border. The 3.0 mgd of combined pumping capacity from the two stations satisfies the projected buildout demand of 2.7 mgd in the 472 Zone. Due to the future pumping and piping configuration through the 433 Zone, the pumping stations serving the 472 Zone would primarily be served by separate supply sources:

- Vista Woods PS would essentially be supplied from Smith Lake WTP through the Moncure PS and water mains along Garrisonville Road.
- Lightfoot Drive PS would be fed from the Abel Lake and Rocky Pen Run WTPs through the pumping station along Courthouse Road at the 370N/433 Zone border.

Expansion of the Moncure PS will be dependent on the expansion at Embrey Mill PS and the projected near-term demands. To fully utilize the available production capacity of Smith Lake WTP (14 mgd), roughly 10.5 mgd will need to be transferred through the Embrey Mill and Moncure Pumping Stations (i.e., 14 mgd production from Smith Lake WTP – 3.5 mgd near-term maximum day demand for 310 Zone). Under the option presented in this Master Plan, the Embrey Mill PS was sized for 2 mgd and the Moncure PS will need to be capable of pumping 8.5 mgd. The configuration of the improvements and flow balance schematics are shown in Appendix A and C of this technical memorandum, respectively.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 433 Zone will be 4.9 mgd. Consequently, the volume of storage needed in the 433 Zone under buildout conditions is roughly 2.5 MG. At buildout, the Amyclae Tank will provide 1.5 MG. Due to the lack of useable storage in the Shelton Shop Standpipe, DOU should consider replacing the standpipe with a 1.0 MG elevated storage tank to satisfy the remaining 1.0 MG deficit. The Shelton Shop elevated tank will provide suction storage for the Vista Woods PS.

Priority	*1 - Critical*
Design	*2020*
Construct	*2021*
Total Project Cost	*$700,000*
Prior Spending	*$0*
Costs in this Plan Period	*$700,000*

433-201: Upgrade and expand Moncure Pumping Station to 8.5 mgd

Smith Lake WTP currently supplies water to three pressure zones with hydraulic grade lines of 310, 433 and 472 feet. Water from the Smith Lake WTP is pumped to the Moncure PS on the western border of the 310 Zone which pumps flow to the 433 Zone. Flow from the 433 Zone is boosted to the 472 Zone through the Vista Woods PS which is located on the western border of the 433 Zone along Shelton Shop Road. The 472 Zone has one elevated tank along Mountain View Road in the vicinity of Spy Glass Lane (0.5 MG Vista Woods Tank at overflow elevation 472 feet).

Under buildout conditions, demands for the 433 Zone (7.3 mgd) and transfers to the 472 Zone (2.7 mgd) will be satisfied by the Moncure PS, 433 Zone PS along Courthouse Road, and the Mountain View PS. As an alternative to construction of the 433 Zone Pumping Station along Courthouse Road and possibly the 18-inch main from Ramoth Church Road to Courthouse Road (370N-03), DOU could consider expanding the 370N Zone PS to include 433 Zone pumps. The 433 Zone pumps could feed the 433 Zone and the future 472 Zone PS on Lightfoot Road near Mountain View Road (472-200) through short segment of 12-inch main connecting the 370N Zone PS to the existing 12-inch main along Mountain View Road. This configuration is not presented as the recommended approach for several reasons:

- The 12-inch main along Mountain View Road is distant from the demand center which is further north in the 433 Zone. Supplying the flow near the demand center improves system reliability.
- The piping network through the 433 Zone from the 12-inch main along Mountain View Road is weak compared with the network connected to the 12-inch main along Courthouse Road.
- The recommended projects create a strong transmission artery between the three treatment plants and along the I-95/Jefferson Davis Highway corridor.

Currently, the Moncure PS is operated off of the water levels in the Shelton Shop Tank. In the near-term future, the Moncure PS would be upgraded and expanded to 8.5 mgd and a second 1.4 mgd pumping station is proposed on the 12-inch main on Lightfoot Drive at the intersection of Mountain View Road at the 472/433 Zone border. The 3.0 mgd of combined pumping capacity from the two stations satisfies the projected buildout demand of 2.7 mgd in the 472 Zone. Due to the future pumping and piping configuration through the 433 Zone, the pumping stations serving the 472 Zone would primarily be served by separate supply sources:

- Vista Woods PS would essentially be supplied from Smith Lake WTP through the Moncure PS and water mains along Garrisonville Road.
- Lightfoot Drive PS would be fed from the Abel Lake and Rocky Pen Run WTPs through the pumping station along Courthouse Road at the 370N/433 Zone border.

Expansion of the Moncure PS will be dependent on the expansion at Embrey Mill PS and the projected near-term demands. To fully utilize the available production capacity of Smith Lake WTP (14 mgd), roughly 10.5 mgd will need to be transferred through the Embrey Mill and Moncure Pumping Stations (i.e., 14 mgd production from Smith Lake WTP – 3.5 mgd near-term maximum day demand for 310 Zone). Under the option presented in this Master Plan, the Embrey Mill PS was sized for 2 mgd and the Moncure PS will need to be capable of pumping 8.5 mgd. The configuration of the improvements and flow balance schematics are shown in Appendix A and C of this technical memorandum, respectively.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 433 Zone will be 4.9 mgd. Consequently, the volume of storage needed in the 433 Zone under buildout conditions is roughly 2.5 MG. At buildout, the Amyclae Tank will provide 1.5 MG. Due to the lack of useable storage in the Shelton Shop Standpipe, DOU should consider replacing the standpipe with a 1.0 MG elevated storage tank to satisfy the remaining 1.0 MG deficit. The Shelton Shop elevated tank will provide suction storage for the Vista Woods PS.

Priority	*1 - Critical*
Design	*2005*
Construct	*2006*
Total Project Cost	*$1,050,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,050,000*

6.9. 472 Zone Improvements

472-100: Construct 0.5 MG storage tank along Garrisonville Road near Ripley Road

Smith Lake WTP currently supplies water to three pressure zones with hydraulic grade lines of 310, 433 and 472 feet. Water from the Smith Lake WTP is pumped to the Moncure PS on the western border of the 310 Zone which pumps flow to the 433 Zone. Flow from the 433 Zone is boosted to the 472 Zone through the Vista Woods PS which is located on the western border of the 433 Zone along Shelton Shop Road. The 472 Zone has one elevated tank along Mountain View Road in the vicinity of Spy Glass Lane (0.5 MG Vista Woods Tank at overflow elevation 472 feet).

Currently, the Vista Woods PS is operated off of the water levels in the Vista Woods Tank. In the future, a second pumping station is proposed on the 12-inch main on Lightfoot Drive at the intersection of Mountain View Road at the 472/433 Zone border. Due to the future pumping and piping configuration through the 433 Zone, the pumping stations serving the 472 Zone would primarily be served by separate supply sources:

- Vista Woods PS would essentially be supplied from Smith Lake WTP through the Moncure PS and water mains along Garrisonville Road.
- Lightfoot Drive PS would be fed from the Abel Lake and Rocky Pen Run WTPs through the pumping station along Courthouse Road at the 370N/433 Zone border.

The configuration of the improvements and flow balance schematics are shown in Appendix A and C of this technical memorandum, respectively.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 472 Zone will be 1.8 mgd. Consequently, the volume of storage needed in the 472 Zone under buildout conditions is roughly 0.9 MG. A second 0.5 MG elevated tank is proposed for the 472 Zone to satisfy the projected storage deficit. The proposed site for the new tank is along Garrisonville Road near Ripley Road. This tank would typically provide storage to the northern portion of the 472 Zone.

Priority	*2 - Necessary*
Design	*2024*
Construct	*2025*
Total Project Cost	*$863,000*
Prior Spending	*$0*
Costs in this Plan Period	*$863,000*

472-200: Construct 1.4 mgd pumping station along Lightfoot Road near Mountain View Road

Smith Lake WTP currently supplies water to three pressure zones with hydraulic grade lines of 310, 433 and 472 feet. Water from the Smith Lake WTP is pumped to the Moncure PS on the western border of the 310 Zone which pumps flow to the 433 Zone. Flow from the 433 Zone is boosted to the 472 Zone through the Vista Woods PS which is located on the western border of the 433 Zone along Shelton Shop Road. The 472 Zone has one elevated tank along Mountain View Road in the vicinity of Spy Glass Lane (0.5 MG Vista Woods Tank at overflow elevation 472 feet).

Currently, the Vista Woods PS is operated off of the water levels in the Vista Woods Tank. In the future, the Vista Woods PS would be upgraded and expanded to 1.6 mgd and a second 1.4 mgd pumping station is proposed on the 12-inch main on Lightfoot Drive at the intersection of Mountain View Road at the 472/433 Zone border. The 3.0 mgd of combined pumping capacity from the two stations satisfies the projected buildout demand of 2.7 mgd in the 472 Zone. Due to the future pumping and piping configuration through the 433 Zone, the pumping stations serving the 472 Zone would primarily be served by separate supply sources:

- Vista Woods PS would essentially be supplied from Smith Lake WTP through the Moncure PS and water mains along Garrisonville Road.
- Lightfoot Drive PS would be fed from the Abel Lake and Rocky Pen Run WTPs through the pumping station along Courthouse Road at the 370N/433 Zone border.

The configuration of the improvements and flow balance schematics are shown in Appendix A and C of this technical memorandum, respectively.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 472 Zone will be 1.8 mgd. Consequently, the volume of storage needed in the 472 Zone under buildout conditions is roughly 0.9 MG. A second 0.5 MG elevated tank is proposed for he 472 Zone to satisfy the projected storage deficit. The proposed site for the new tank is along Garrisonville Road near Ripley Road. This tank would typically provide storage to the northern portion of the 472 Zone.

Priority	*1 - Critical*
Design	*2020*
Construct	*2021*
Total Project Cost	*$350,000*
Prior Spending	*$0*
Costs in this Plan Period	*$350,000*

<u>472-201: Upgrade and expand Vista Woods Pumping Station to 1.6 mgd</u>

Smith Lake WTP currently supplies water to three pressure zones with hydraulic grade lines of 310, 433 and 472 feet. Water from the Smith Lake WTP is pumped to the Moncure PS on the western border of the 310 Zone which pumps flow to the 433 Zone. Flow from the 433 Zone is boosted to the 472 Zone through the Vista Woods PS which is located on the western border of the 433 Zone along Shelton Shop Road. The 472 Zone has one elevated tank along Mountain View Road in the vicinity of Spy Glass Lane (0.5 MG Vista Woods Tank at overflow elevation 472 feet).

Currently, the Vista Woods PS is operated off of the water levels in the Vista Woods Tank. In the future, the Vista Woods PS would be upgraded and expanded to 1.6 mgd and a second 1.4 mgd pumping station is proposed on the 12-inch main on Lightfoot Drive at the intersection of Mountain View Road at the 472/433 Zone border. The 3.0 mgd of combined pumping capacity from the two stations satisfies the projected buildout demand of 2.7 mgd in the 472 Zone. Due to the future pumping and piping configuration through the 433 Zone, the pumping stations serving the 472 Zone would primarily be served by separate supply sources:

- Vista Woods PS would essentially be supplied from Smith Lake WTP through the Moncure PS and water mains along Garrisonville Road.
- Lightfoot Drive PS would be fed from the Abel Lake and Rocky Pen Run WTPs through the pumping station along Courthouse Road at the 370N/433 Zone border.

The configuration of the improvements and flow balance schematics are shown in Appendix A and C of this technical memorandum, respectively.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 472 Zone will be 1.8 mgd. Consequently, the volume of storage needed in the 472 Zone under buildout conditions is roughly 0.9 MG. A second 0.5 MG elevated tank is proposed for he 472 Zone to satisfy the projected storage deficit. The proposed site for the new tank is along Garrisonville Road near Ripley Road. This tank would typically provide storage to the northern portion of the 472 Zone.

Priority	*1 - Critical*
Design	*2007*
Construct	*2008*
Total Project Cost	*$480,000*
Prior Spending	*$0*
Costs in this Plan Period	*$480,000*

6.10. 480 Zone Improvements

<u>480-01: Construct 24-inch main from Rocky Pen Run WTP to existing 12-inch and proposed 18-inch mains near Good Neighbor Lane (4,993 feet)</u>

This project involves design and construction of a 24-inch main from the proposed Rocky Pen Run WTP near Burgess Lane to the existing 12-inch and proposed 18-inch water mains near Good Neighbor Lane (4,993 feet). The purpose of the project is to convey flows from Rocky Pen Run WTP to the 480 Zone and the 520 Zone. Construction of the water main should be concurrent with construction of the Rocky Pen Run WTP.

Priority	*1 - Critical*
Design	*2012*
Construct	*2013*
Total Project Cost	*$809,000*
Prior Spending	*$0*
Costs in this Plan Period	*$809,000*

<u>480-02: Construct 12-inch main along Truslow Road from Berea Church Road to Norfolk Street (4,856 feet)</u>

This project involves design and construction of a 12-inch main along Truslow Road from the 12-inch main on Berea Church Road to the 12-inch main on Norfolk Street (4,856 feet). The purpose of the project is to provide a connection between the existing 12-inch mains to create a loop to enhance reliability and increase conveyance capacity in the existing 503 Zone and the future 480 Zone.

Priority	*3 – Prior Appropriation*
Design	*Not Applicable*
Construct	*Not Applicable*
Total Project Cost	*$441,000*
Prior Spending	*$0*
Costs in this Plan Period	*$441,000*

<u>480-03: Construct 18-inch main from proposed 24-inch near Good Neighbor Lane to proposed 520 Zone Pumping Station near Estes Lane (6,500 feet)</u>

This project involves design and construction of an 18-inch main from the proposed 24-inch near Good Neighbor Lane to proposed 520 Zone Pumping Station near Estes Lane (6,500 feet). The purpose of the project is to convey flow from the proposed 24-inch main from Rocky Pen Run WTP to the 480 Zone and 520 Zone.

Priority	*1 - Critical*
Design	*2012*
Construct	*2013*
Total Project Cost	*$811,000*
Prior Spending	*$0*
Costs in this Plan Period	*$811,000*

<u>480-04: Construct 12-inch main along University Boulevard from existing 12-inch main on Reservoir Road to existing 12-inch main on University Boulevard (930 feet)</u>

This project involves design and construction of a 12-inch main along University Boulevard from the existing 12-inch main on Reservoir Road to the existing 12-inch main on University Boulevard (930 feet). The lower elevations on the southern portion of Stafford Lakes Village are planned to be part of the 480 Zone via construction of a short segment of 12-inch main along University Boulevard (480-04). Due to high ground elevations, the area in the vicinity of University of Mary Washington/James Monroe Center for Graduate and Professional Studies will be part of the 520 Zone and will be served by transmission mains along Warrenton Road and Village Parkway. A PRV is recommended on the 12-inch main along Village Parkway at the 520/480 Zone border to serve as a backup supply to the 480 Zone portion of Stafford Lakes Village.

Priority	*1 - Critical*
Design	*2012*
Construct	*2013*
Total Project Cost	*$84,000*
Prior Spending	*$0*
Costs in this Plan Period	*$84,000*

480-100: Construct 1.0 MG elevated storage tank along Greenbank Road in vicinity of Good Neighbor Lane

Abel Lake WTP currently supplies water to two pressure zones with hydraulic grade lines of 342 feet and 503 feet. Water from the Abel Lake WTP is pumped through 16-inch water main to the Abel Lake Tank. The Abel Lake Tank is a 4 MG ground level tank with an overflow elevation of 298 feet. Water is pumped from the Abel Lake Tank to the 342 Zone through the Cranes Corner Pumping Station and to the 503 Zone through the Berea Pumping Station. Finished water storage in the 503 Zone is provided by the 0.5 MG Berea Tank which has an overflow elevation of 503 feet.

Currently, the Berea Pumping Station is operated off of the water levels in the Berea Tank. After Rocky Pen Run Reservoir WTP and the transmission mains to Warrenton Road are completed, the existing Berea Tank (overflow elevation 503 ft) will eventually be eliminated and the 503 Zone will be split into two pressure zones: 520 Zone and 480 Zone. The 520 Zone will be established to satisfy low pressure problems at the higher ground elevations along Warrenton Road west of Estes Road while maintaining acceptable operating pressures at the lower elevations in the eastern portion of the existing 503 Zone by dropping the hydraulic grade in this area to 480 feet. A new 24-inch water main from Rocky Pen Run WTP will be extended to the transmission mains on Warrenton Road to supply the 480 Zone. The 480 Zone will include a new 1.0 MG elevated tank along Greenbank Road in the vicinity of Good Neighbor Lane. Two PRVs along the southern border of the 480 Zone are proposed to serve the 370S Zone: one PRV on the transmission main along Virginia Parkway and one PRV on the 12-inch main on Sanford Drive near Warrenton Road. A new 3.4 mgd pumping station will be constructed on the transmission mains along Warrenton Road near Estes Lane to pump the 520 Zone maximum day demand (3.4 mgd) from the 480 Zone. The lower elevations on the southern portion of Stafford Lakes Village are planned to be part of the 480 Zone via construction of a short segment of 12-inch main along University Boulevard (480-04) from the 24-inch main between Rocky Pen Run WTP and Warrenton Road and the existing 12-inch main on University Boulevard. A PRV is recommended on the 12-inch main along Village Parkway to serve as a backup supply to Stafford Lakes Village. A new 1.0 MG elevated tank in the 520 Zone will be used to control the pumps at the Warrenton Road PS that feed the 520 Zone. Two PRVs are proposed to serve the 370S Zone from the 480 Zone: one PRV on the transmission main along Virginia Parkway and one PRV on the 12-inch main on Sanford Drive near Warrenton Road.

The configuration of the improvements and flow balance schematics are shown in Appendix A and C of this technical memorandum, respectively.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 480 Zone will be 2.9 mgd. Consequently, the volume of storage needed in the 480 Zone under buildout conditions is roughly 1.5 MG. A new 1.0 MG elevated storage tank is proposed along Greenbank Road in the vicinity of Good Neighbor Lane. Due to the strong connection through the transmission main along the Virginia Parkway, 0.5 MG of storage in the 1.0 MG tank along Greenbank Road is allocated to the 370S Zone. The remaining 0.5 MG of storage in the 1.0 MG tank along Greenbank Road is assigned to the 480 Zone leaving a 1.0 MG storage deficit. Due to the lack of promising storage tank sites in the 480 Zone, 1.0 MG of storage at the Rocky Pen Run WTP will be assigned to the 480 Zone to satisfy the remaining projected storage deficit. Pumping facilities at Rocky Pen Run WTP should be sized to deliver the 1.0 MG of storage to the 480 Zone.

Priority	*1 - Critical*
Design	*2011*
Construct	*2012*
Total Project Cost	*$1,725,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,725,000*

6.11. 520 Zone Improvements

<u>520-01: Construct 12-inch main along Warrenton Road from 520 Zone Pumping Station near Estes Lane to Village Parkway (1,375 feet)</u>

This project involves design and construction of a 12-inch main along Warrenton Road from the Warrenton Road PS near Estes Lane to Village Parkway (1,375 feet). The purpose of the project is to convey flows from the Warrenton Road PS to the proposed 18-inch main along Warrenton Road at Village Parkway which supplies the Westlake Industrial Park area that is currently not served with public water or sewer service. Construction of the water main should be concurrent with establishment of the 520 Zone (i.e., construction of the Warrenton Road PS and storage tank at Westlake Industrial Park). In addition, the timing for construction of the water main should be consistent with construction of the sewer infrastructure in this area which is proposed in the 2015 timeframe.

Priority	*1 - Critical*
Design	*2013*
Construct	*2014*
Total Project Cost	*$125,000*
Prior Spending	*$0*
Costs in this Plan Period	*$125,000*

<u>520-02: Construct 18-inch main along Warrenton Road from Village Parkway to Poplar Road (5,477 feet)</u>

This project involves design and construction of an 18-inch main along Warrenton Road from Village Parkway to Poplar Road (5,477 feet). The purpose of the project is to convey flows along Warrenton Road to the Westlake Industrial Park area that is currently not served with public water or sewer service. Construction of the water main should be concurrent with establishment of the 520 Zone (i.e., construction of the Warrenton Road PS and storage tank at Westlake Industrial Park). In addition, the timing for construction of the water main should be consistent with construction of the sewer infrastructure in this area which is proposed in the 2015 timeframe.

Priority	*1 - Critical*
Design	*2013*
Construct	*2014*
Total Project Cost	*$683,000*
Prior Spending	*$0*
Costs in this Plan Period	*$683,000*

<u>520-03: Construct 18-inch main along Warrenton Road from Poplar Road to Clark Patton Road (1,000 feet)</u>

This project involves design and construction of a 16-inch main along Clark Patton Road from the proposed 18-inch along Warrenton Road to the proposed water storage tank along Clark Patton Road (1,000 feet). The purpose of the project is to convey flows from the proposed 18-inch main along Warrenton Road to the proposed storage tank along Clark Patton Road. This project supplies flow to the Westlake Industrial Park area that is currently not served with public water or sewer service. Construction of the water main should be concurrent with establishment of the 520 Zone (i.e., construction of the Warrenton Road PS and storage tank at Westlake Industrial Park). In addition, the timing for construction of the water main should be consistent with construction of the sewer infrastructure in this area which is proposed in the 2015 timeframe.

Priority	*1 - Critical*
Design	*2013*

Construct	*2014*
Total Project Cost	*$125,000*
Prior Spending	*$0*
Costs in this Plan Period	*$125,000*

520-04: Construct 16-inch main along Clark Patton Road from Warrenton Road to Westlake Tank (3,000 feet)

This project involves design and construction of a 16-inch main along Clark Patton Road from the proposed 18-inch main along Warrenton Road to the proposed water storage tank along Clark Patton Road (3,000 feet). The purpose of the project is to convey flows from the proposed 18-inch main along Warrenton Road to the proposed storage tank along Clark Patton Road. This project supplies flow to the Westlake Industrial Park area that is currently not served with public water or sewer service. Construction of the water main should be concurrent with establishment of the 520 Zone (i.e., construction of the Warrenton Road PS and storage tank at Westlake Industrial Park). In addition, the timing for construction of the water main should be consistent with construction of the sewer infrastructure in this area which is proposed in the 2015 timeframe.

Priority	*4 - Developer*
Design	*Not Applicable*
Construct	*Not Applicable*
Total Project Cost	*$340,000*
Prior Spending	*$0*
Costs in this Plan Period	*$340,000*

520-100: Construct 1.0 MG elevated storage tank along Warrenton Road in vicinity of Clark Patton Road

Abel Lake WTP currently supplies water to two pressure zones with hydraulic grade lines of 342 feet and 503 feet. Water from the Abel Lake WTP is pumped through 16-inch water main to the Abel Lake Tank. The Abel Lake Tank is a 4 MG ground level tank with an overflow elevation of 298 feet. Water is pumped from the Abel Lake Tank to the 342 Zone through the Cranes Corner Pumping Station and to the 503 Zone through the Berea Pumping Station. Finished water storage in the 503 Zone is provided by the 0.5 MG Berea Tank which has an overflow elevation of 503 feet.

Currently, the Berea Pumping Station is operated off of the water levels in the Berea Tank. After Rocky Pen Run Reservoir WTP and the transmission mains to Warrenton Road are completed, the existing Berea Tank (overflow elevation 503 ft) will eventually be eliminated and the 503 Zone will be split into two pressure zones: 520 Zone and 480 Zone. The 520 Zone will be established to satisfy low pressure problems at the higher ground elevations along Warrenton Road west of Estes Road while maintaining acceptable operating pressures at the lower elevations in the eastern portion of the existing 503 Zone by dropping the hydraulic grade in this area to 480 feet. A new 24-inch water main from Rocky Pen Run WTP will be extended to the transmission mains on Warrenton Road to supply the 480 Zone. The 480 Zone will include a new 1.0 MG elevated tank along Greenbank Road in the vicinity of Good Neighbor Lane. Two PRVs along the southern border of the 480 Zone are proposed to serve the 370S Zone: one PRV on the transmission main along Virginia Parkway and one PRV on the 12-inch main on Sanford Drive near Warrenton Road. A new 3.4 mgd pumping station will be constructed on the transmission mains along Warrenton Road near Estes Lane to pump the 520 Zone maximum day demand (3.4 mgd) from the 480 Zone. The lower elevations on the southern portion of Stafford Lakes Village are planned to be part of the 480 Zone via construction of a short segment of 12-inch main along University Boulevard (480-04) from the 24-inch main between Rocky Pen Run WTP and Warrenton Road and the existing 12-inch main on University Boulevard. A PRV is recommended on the 12-inch main along Village Parkway to serve as a backup supply to Stafford Lakes Village. A new 1.0 MG elevated tank in the 520 Zone will be used to control the pumps at the Warrenton Road PS that feed the 520 Zone.

The configuration of the improvements and flow balance schematics are shown in Appendix A and C of this technical memorandum, respectively.

The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 520 Zone will be 2.3 mgd. Consequently, the volume of storage needed in the 520 Zone under buildout conditions is roughly 1.1 MG. A new 1.0 MG elevated storage tank is proposed along Warrenton Road near Estes Lane to satisfy the projected storage deficit. Construction of the storage tank should be concurrent with establishment of the 520 Zone (i.e., construction of the Warrenton Road PS and transmission mains). In addition, the timing for construction of the storage tank should be consistent with construction of the infrastructure in this area, such as the sewer facilities which are proposed in the 2015 timeframe.

Priority	*4 - Developer*
Design	*Not Applicable*
Construct	*Not Applicable*
Total Project Cost	*$1,725,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,725,000*

520-200: Construct 3.4 mgd pumping station along Warrenton Road near Estes Lane

Abel Lake WTP currently supplies water to two pressure zones with hydraulic grade lines of 342 feet and 503 feet. Water from the Abel Lake WTP is pumped through 16-inch water main to the Abel Lake Tank. The Abel Lake Tank is a 4 MG ground level tank with an overflow elevation of 298 feet. Water is pumped from the Abel Lake Tank to the 342 Zone through the Cranes Corner Pumping Station and to the 503 Zone through the Berea Pumping Station. Finished water storage in the 503 Zone is provided by the 0.5 MG Berea Tank which has an overflow elevation of 503 feet.

Currently, the Berea Pumping Station is operated off of the water levels in the Berea Tank. After Rocky Pen Run Reservoir WTP and the transmission mains to Warrenton Road are complete, the existing Berea Tank (overflow elevation 503 ft) will eventually be eliminated and the 503 Zone will be split into two pressure zones: 520 Zone and 480 Zone. The 520 Zone will be established to satisfy low pressure problems at the higher ground elevations along Warrenton Road west of Estes Road while maintaining acceptable operating pressures at the lower elevations in the eastern portion of the existing 503 Zone by dropping the hydraulic grade in this area to 480 feet. A new 24-inch water main from Rocky Pen Run WTP will be extended to the transmission mains on Warrenton Road to supply the 480 Zone. The 480 Zone will include a new 1.0 MG elevated tank along Greenbank Road in the vicinity of Good Neighbor Lane. Two PRVs along the southern border of the 480 Zone are proposed to serve the 370S Zone: one PRV on the transmission main along Virginia Parkway and one PRV on the 12-inch main on Sanford Drive near Warrenton Road. A new 3.4 mgd pumping station will be constructed on the transmission mains along Warrenton Road near Estes Lane to pump the 520 Zone maximum day demand (3.4 mgd) from the 480 Zone. The lower elevations on the southern portion of Stafford Lakes Village are planned to be part of the 480 Zone via construction of a short segment of 12-inch main along University Boulevard (480-04) from the 24-inch main between Rocky Pen Run WTP and Warrenton Road and the existing 12-inch main on University Boulevard. A PRV is recommended on the 12-inch main along Village Parkway to serve as a backup supply to Stafford Lakes Village. A new 1.0 MG elevated tank in the 520 Zone will be used to control the pumps at the Warrenton Road PS that feed the 520 Zone.

The configuration of the improvements and flow balance schematics are shown in Appendix A and C of this technical memorandum, respectively.

As an alternative to construction of the Warrenton Road PS, an 18-inch transmission main from Rocky Pen Run WTP could be extended to Warrenton Road to directly feed the 520 Zone. An advantage of this alternative is the centralized pumping at Rocky Pen Run WTP.

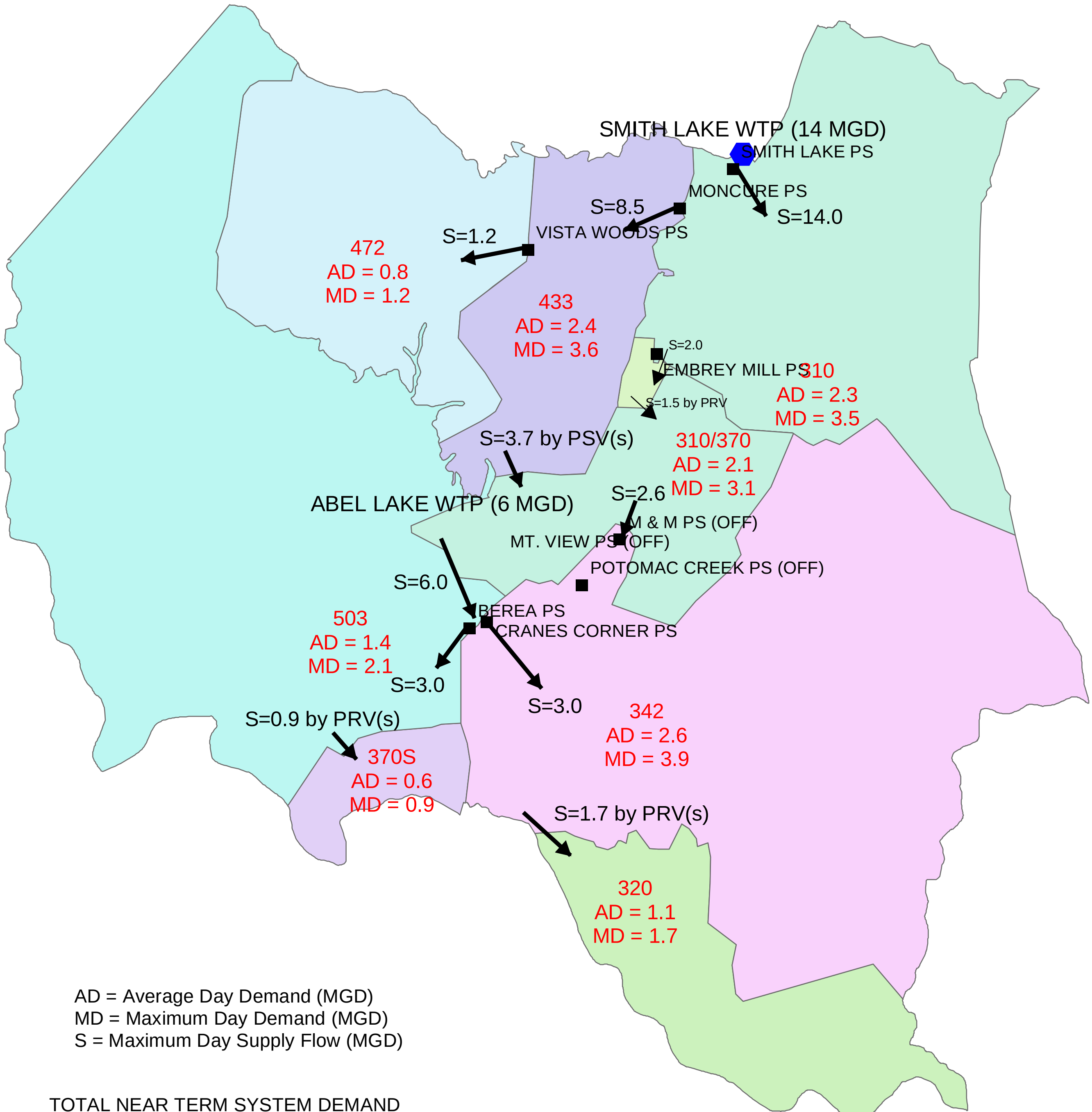
The adequacy of storage for each pressure zone was assessed using the required volume of effective storage equal to one-half of the average day demand in accordance with VDH requirements. The average day demand under buildout conditions for the 520 Zone will be 2.3 mgd. Consequently, the volume of storage needed in the 520 Zone under buildout conditions is roughly 1.1 MG. A new 1.0 MG elevated storage tank is proposed along Warrenton Road near Estes Lane to satisfy the projected storage deficit. Construction of the storage tank should be concurrent with establishment of the 520 Zone (i.e., construction of the Warrenton Road PS and transmission mains). In addition, the timing for construction of the storage tank should be consistent with construction of the infrastructure in this area, such as the sewer facilities which are proposed in the 2015 timeframe.

Priority	*1 - Critical*
Design	*2013*
Construct	*2014*
Total Project Cost	*$680,000*
Prior Spending	*$0*
Costs in this Plan Period	*$680,000*

Appendix A

Flow Balance Diagrams

STAFFORD COUNTY
WATER SYSTEM NEAR-TERM SCENARIO
(PRIOR TO ROCKYPEN RUN WTP)
MAX DAY FLOW BALANCE (OPTION 1)



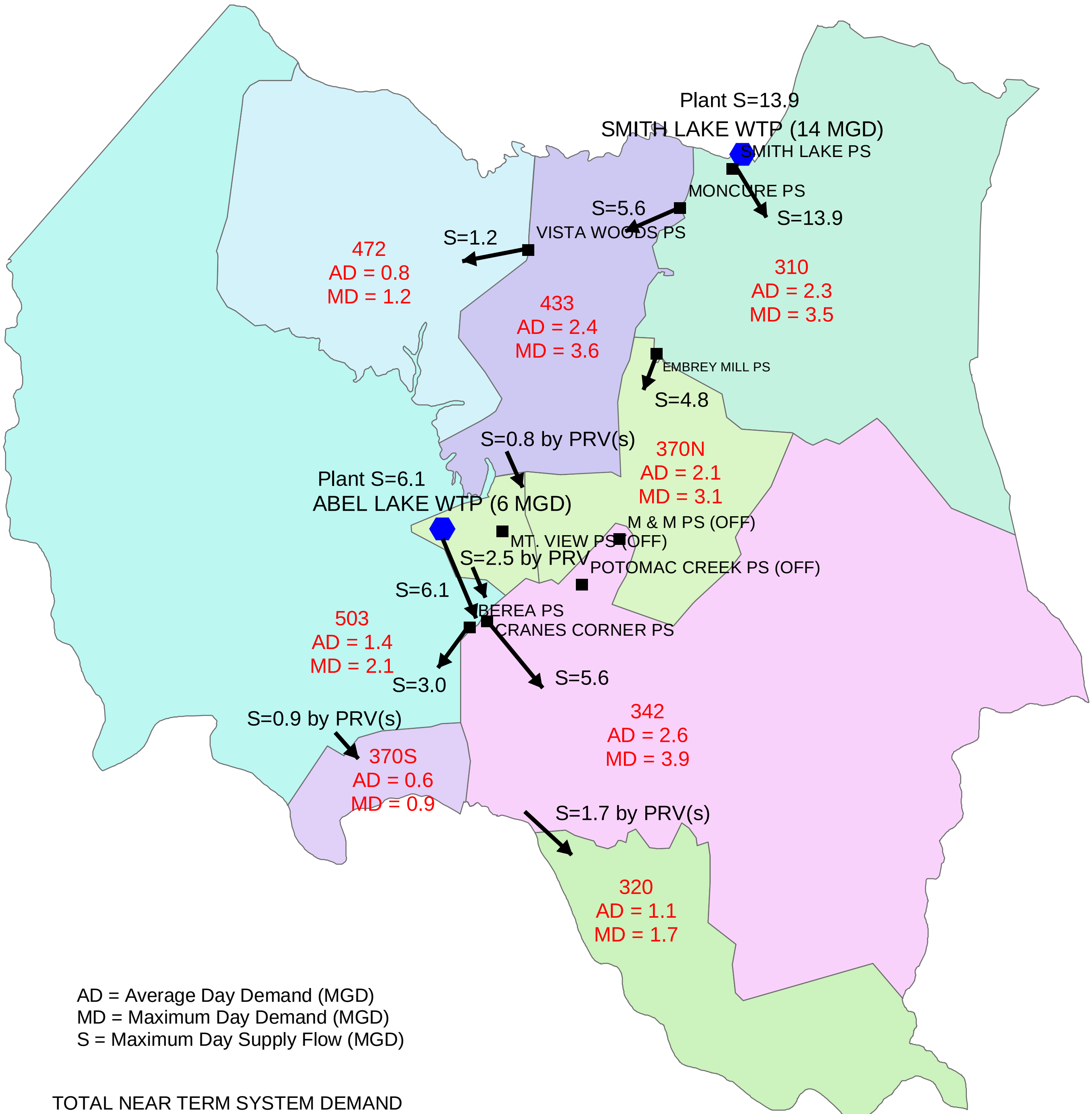
AD = Average Day Demand (MGD)
MD = Maximum Day Demand (MGD)
S = Maximum Day Supply Flow (MGD)

TOTAL NEAR TERM SYSTEM DEMAND
AD = 13.3 MGD
MD = 20.0 MGD

O'BRIEN & GERE ENGINEERS, INC.

SEPTEMBER 2003

STAFFORD COUNTY
WATER SYSTEM NEAR TERM SCENARIO
(PRIOR TO ROCKYPEN RUN WTP)
MAX DAY FLOW BALANCE (OPTION 2)



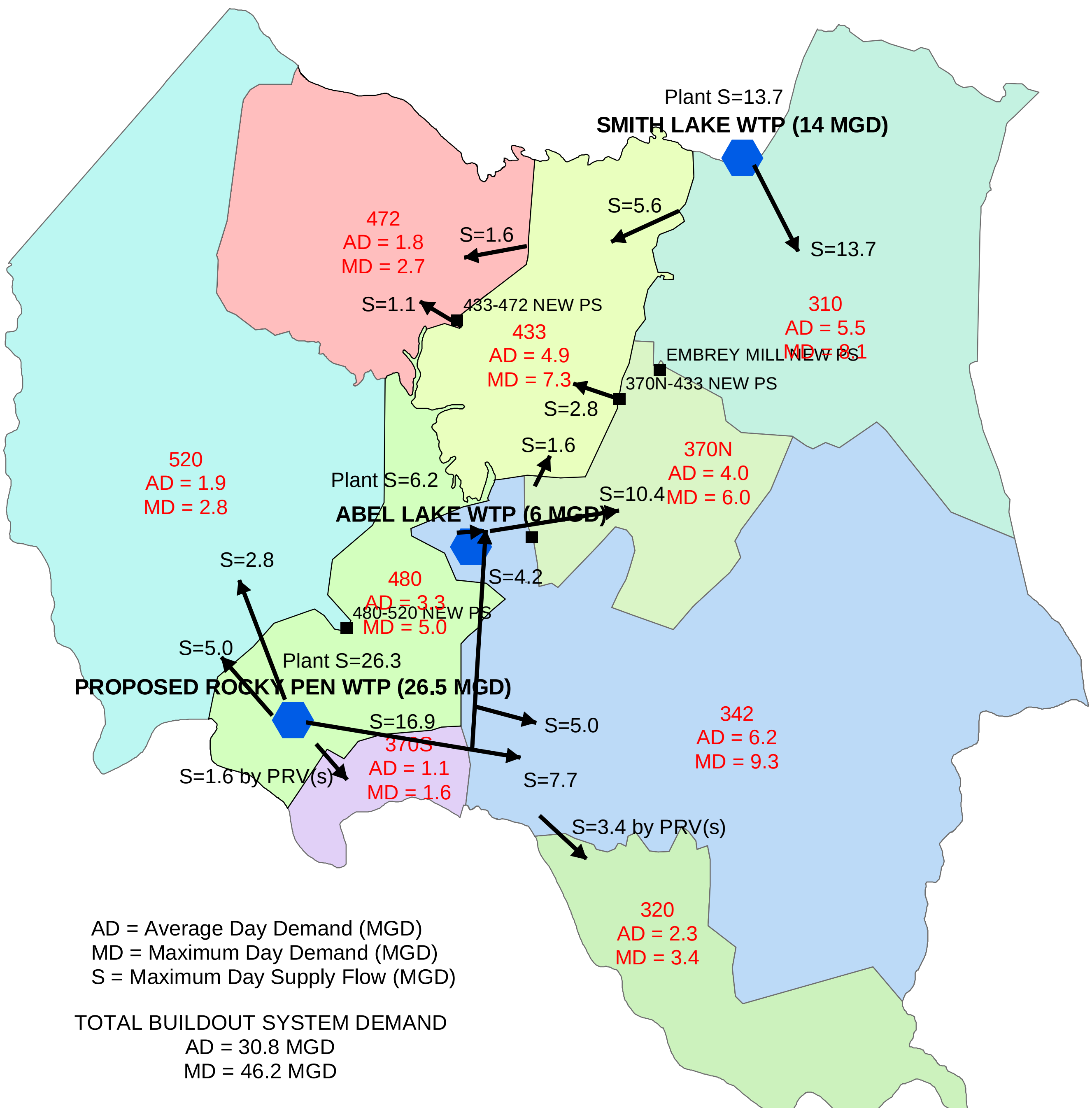
AD = Average Day Demand (MGD)
MD = Maximum Day Demand (MGD)
S = Maximum Day Supply Flow (MGD)

TOTAL NEAR TERM SYSTEM DEMAND
AD = 13.3 MGD
MD = 20.0 MGD

O'BRIEN & GERE ENGINEERS, INC.

SEPTEMBER 2003

STAFFORD COUNTY
WATER SYSTEM BUILDOUT SCENARIO
MAX DAY FLOW BALANCE PROPOSAL



AD = Average Day Demand (MGD)
MD = Maximum Day Demand (MGD)
S = Maximum Day Supply Flow (MGD)

TOTAL BUILDOUT SYSTEM DEMAND
AD = 30.8 MGD
MD = 46.2 MGD

*HIGHER HYDRAULIC GRADIENTS THAN 433 FT WILL BE AVAILABLE IN THE WEST 370N ZONE.

NOTE:
AN ELEVATED STORAGE TANK (2 MG, 345 FT OVERFLOW) AND A PUMP STATION ARE PROPOSED IN THE 370N ZONE FOR TRANSMITTING FLOWS.
THE EXISTING ABEL LAKE 4 MG GROUND STORAGE TANK WILL BE ABANDONED.

O'BRIEN & GERE ENGINEERS, INC.

Stafford County Water and Sewer Master Plan

4/27/2004

Water Demand by Pressure Zone

	System-wide Maximum Day Demand (mgd)	310 Zone Maximum Day Demand (mgd)	433 Zone Maximum Day Demand (mgd)	472 Zone Maximum Day Demand (mgd)	370N Zone Maximum Day Demand (mgd)	342 Zone Maximum Day Demand (mgd)	320 Zone Maximum Day Demand (mgd)	370S Zone Maximum Day Demand (mgd)	503 Zone Maximum Day Demand (mgd)	Check on System-wide Maximum Day Demand (mgd)
1998										
1999	10.3	1.8	1.6	0.6	1.3	2.1	0.8	0.4	1.7	10.3
2000	11	1.9	1.7	0.6	1.4	2.2	0.8	0.4	1.9	11.0
2001	11.7	2.0	1.8	0.7	1.5	2.4	0.9	0.4	2.0	11.7
2002	12.4	2.2	2.0	0.7	1.6	2.5	0.9	0.4	2.1	12.4
2003	13.1	2.3	2.1	0.8	1.7	2.6	1.0	0.5	2.2	13.1
2004	13.8	2.4	2.2	0.8	1.8	2.8	1.0	0.5	2.3	13.8
2005	14.5	2.5	2.3	0.8	1.9	2.9	1.1	0.5	2.4	14.5
2006	15.2	2.7	2.4	0.9	2.0	3.1	1.1	0.5	2.6	15.2
2007	15.9	2.8	2.5	0.9	2.1	3.2	1.2	0.6	2.7	15.9
2008	16.6	2.9	2.6	1.0	2.2	3.3	1.2	0.6	2.8	16.6
2009	17.3	3.0	2.7	1.0	2.2	3.5	1.3	0.6	2.9	17.3
2010	18.1	3.2	2.8	1.1	2.3	3.6	1.3	0.6	3.0	18.0
2011	18.8	3.3	3.0	1.1	2.4	3.8	1.4	0.6	3.2	18.7
2012	19.5	3.4	3.1	1.1	2.5	3.9	1.4	0.7	3.3	19.4
2013	20.2	3.5	3.2	1.2	2.6	4.1	1.5	0.7	3.4	20.1
2014	20.9	3.7	3.3	1.2	2.7	4.2	1.5	0.7	3.5	20.8
2015	21.6	3.8	3.4	1.3	2.8	4.3	1.6	0.7	3.6	21.5
2016	22.3	3.9	3.5	1.3	2.9	4.5	1.6	0.8	3.8	22.2
2017	23.0	4.0	3.6	1.3	3.0	4.6	1.7	0.8	3.9	22.9
2018	23.7	4.1	3.7	1.4	3.1	4.8	1.7	0.8	4.0	23.6
2019	24.4	4.3	3.8	1.4	3.2	4.9	1.8	0.8	4.1	24.4
2020	25.1	4.4	4.0	1.5	3.3	5.0	1.8	0.9	4.2	25.1
2021	25.8	4.5	4.1	1.5	3.3	5.2	1.9	0.9	4.3	25.8
2022	26.5	4.6	4.2	1.5	3.4	5.3	1.9	0.9	4.5	26.5
2023	27.2	4.8	4.3	1.6	3.5	5.5	2.0	0.9	4.6	27.2
2024	27.9	4.9	4.4	1.6	3.6	5.6	2.1	1.0	4.7	27.9
2025	28.6	5.0	4.5	1.7	3.7	5.8	2.1	1.0	4.8	28.6
2026	29.4	5.1	4.6	1.7	3.8	5.9	2.2	1.0	4.9	29.3
2027	30.1	5.3	4.7	1.8	3.9	6.0	2.2	1.0	5.1	30.0
2028	30.8	5.4	4.9	1.8	4.0	6.2	2.3	1.1	5.2	30.7
2029	31.5	5.5	5.0	1.8	4.1	6.3	2.3	1.1	5.3	31.4
2030	32.2	5.6	5.1	1.9	4.2	6.5	2.4	1.1	5.4	32.1
2031	32.9	5.8	5.2	1.9	4.3	6.6	2.4	1.1	5.5	32.8
2032	33.6	5.9	5.3	2.0	4.4	6.7	2.5	1.2	5.7	33.5
2033	34.3	6.0	5.4	2.0	4.4	6.9	2.5	1.2	5.8	34.2
2034	35.0	6.1	5.5	2.0	4.5	7.0	2.6	1.2	5.9	34.9
2035	35.7	6.2	5.6	2.1	4.6	7.2	2.6	1.2	6.0	35.6
2036	36.4	6.4	5.7	2.1	4.7	7.3	2.7	1.3	6.1	36.3
2037	37.1	6.5	5.9	2.2	4.8	7.5	2.7	1.3	6.3	37.0
2038	37.8	6.6	6.0	2.2	4.9	7.6	2.8	1.3	6.4	37.7
2039	38.5	6.7	6.1	2.2	5.0	7.7	2.8	1.3	6.5	38.4
2040	39.2	6.9	6.2	2.3	5.1	7.9	2.9	1.4	6.6	39.2
2041	39.9	7.0	6.3	2.3	5.2	8.0	2.9	1.4	6.7	39.9
2042	40.7	7.1	6.4	2.4	5.3	8.2	3.0	1.4	6.8	40.6
2043	41.4	7.2	6.5	2.4	5.4	8.3	3.0	1.4	7.0	41.3
2044	42.1	7.4	6.6	2.5	5.5	8.4	3.1	1.5	7.1	42.0
2045	42.8	7.5	6.7	2.5	5.5	8.6	3.1	1.5	7.2	42.7
2046	43.5	7.6	6.9	2.5	5.6	8.7	3.2	1.5	7.3	43.4
2047	44.2	7.7	7.0	2.6	5.7	8.9	3.2	1.5	7.4	44.1
2048	44.9	7.9	7.1	2.6	5.8	9.0	3.3	1.6	7.6	44.8
2049	45.6	8.0	7.2	2.7	5.9	9.2	3.3	1.6	7.7	45.5
2050	46.3	8.1	7.3	2.7	6.0	9.3	3.4	1.6	7.8	46.2

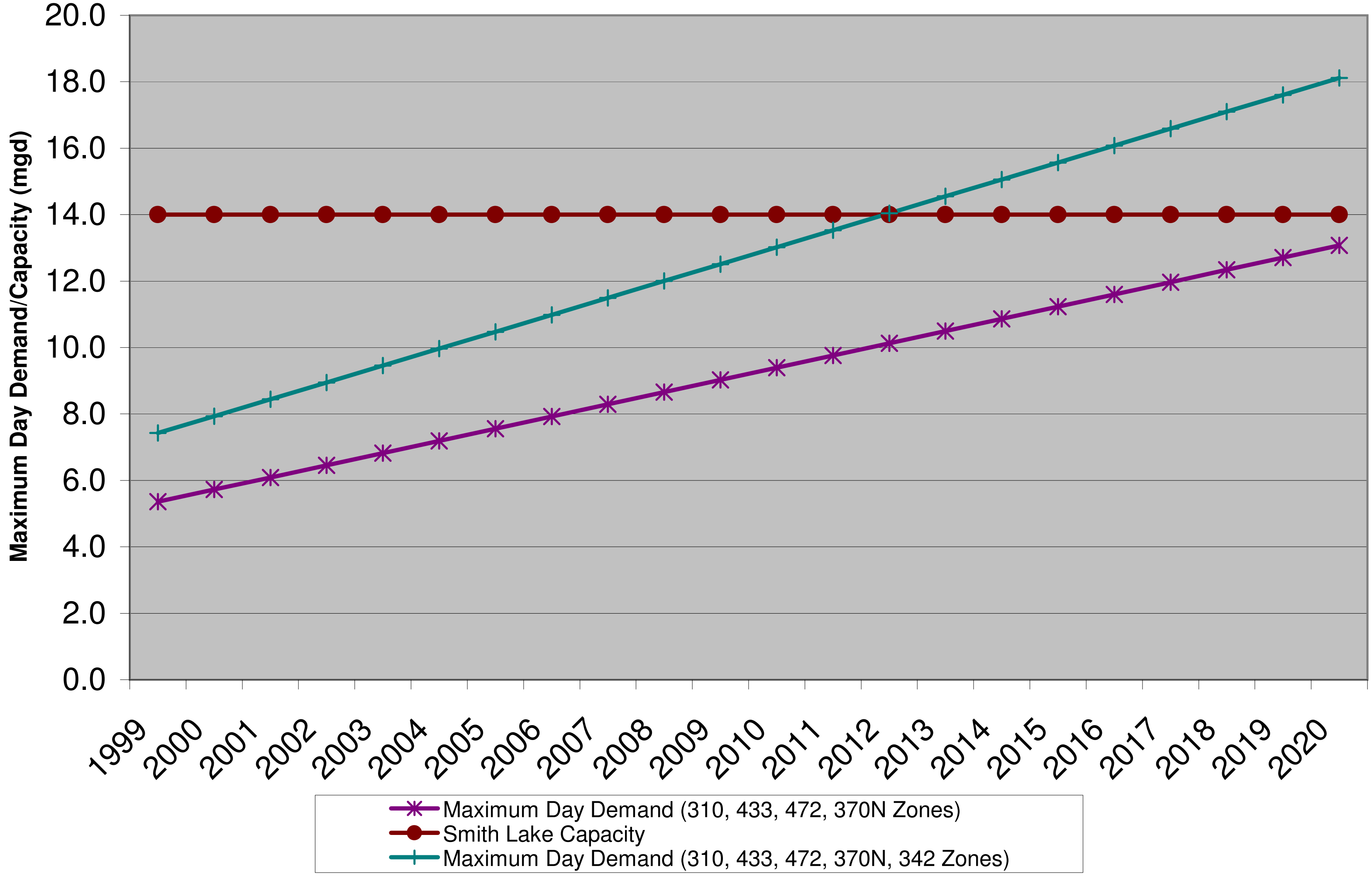
WTP Capacity and Water Demands

	Maximum Day Demand Assigned to Smith Lake WTP (mgd)	Smith Lake WTP Capacity (mgd)	Maximum Day Demand Assigned to Abel Lake WTP (mgd)	Abel Lake WTP Capacity (mgd)	Modified Maximum Day Demand Assigned to Smith Lake WTP (mgd)	Modified Maximum Day Demand Assigned to Abel Lake WTP (mgd)
1999	5.4	14	4.9	6	7.4	2.8
2000	5.7	14	5.3	6	7.9	3.0
2001	6.1	14	5.6	6	8.4	3.2
2002	6.5	14	5.9	6	8.9	3.4
2003	6.8	14	6.3	6	9.5	3.6
2004	7.2	14	6.6	6	10.0	3.8
2005	7.6	14	6.9	6	10.5	4.0
2006	7.9	14	7.3	6	11.0	4.2
2007	8.3	14	7.6	6	11.5	4.4
2008	8.7	14	7.9	6	12.0	4.6
2009	9.0	14	8.3	6	12.5	4.8
2010	9.4	14	8.6	6	13.0	5.0
2011	9.8	14	9.0	6	13.5	5.2
2012	10.1	14	9.3	6	14.0	5.4
2013	10.5	14	9.6	6	14.5	5.6
2014	10.9	14	10.0	6	15.1	5.8
2015	11.2	14	10.3	6	15.6	6.0
2016	11.6	14	10.6	6	16.1	6.2
2017	12.0	14	11.0	6	16.6	6.4
2018	12.3	14	11.3	6	17.1	6.6
2019	12.7	14	11.6	6	17.6	6.7
2020	13.1	14	12.0	6	18.1	6.9

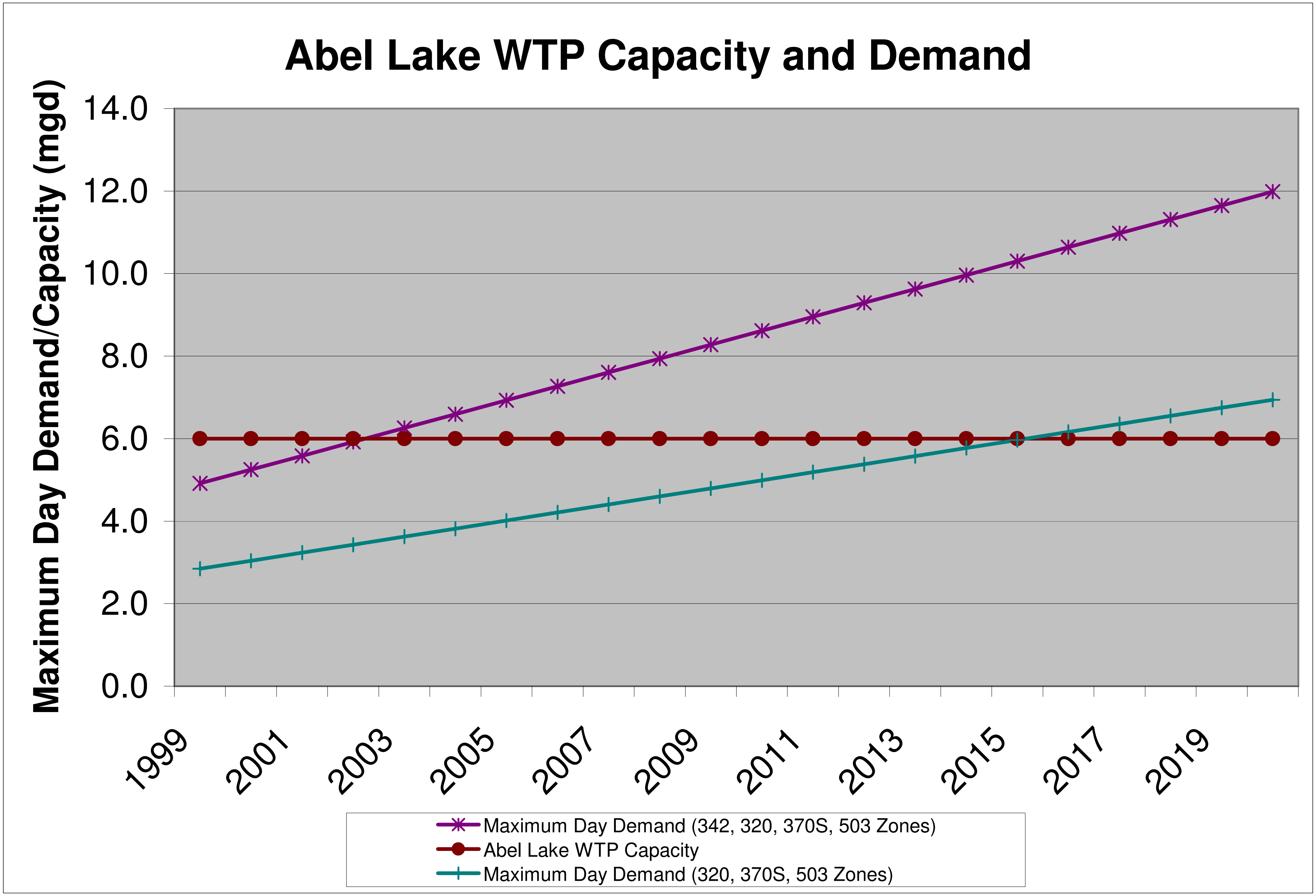
Pumping Station Capacity and Water Demands

	Vista Woods PS Capacity (mgd)	Moncure PS Capacity (mgd)	Maximum Day Demand Assigned to Moncure PS (mgd)	Cranes Corner PS Capacity (mgd)	Maximum Day Demand Assigned to Cranes Corner PS (mgd)	Berea PS Capacity (mgd)	Maximum Day Demand Assigned to Berea PS (mgd)	Embrey Mill PS Capacity (mgd)	Maximum Day Demand Assigned to Embrey Mill PS (mgd)
1999	1.0	5.0	2.4	5.2	2.8	3.2	2.1	2.0	1.8
2000	1.0	5.0	2.7	5.2	3.0	3.2	2.2	2.0	1.9
2001	1.0	5.0	3.1	5.2	3.2	3.2	2.4	2.0	2.0
2002	1.0	5.0	3.4	5.2	3.4	3.2	2.5	2.0	2.0
2003	1.0	5.0	3.7	5.2	3.6	3.2	2.7	2.0	2.0
2004	1.0	5.0	4.1	5.2	3.8	3.2	2.8	2.0	2.0
2005	1.0	5.0	4.4	5.2	4.0	3.2	2.9	2.0	2.0
2006	1.0	5.0	4.7	5.2	4.2	3.2	3.1	2.0	2.0
2007	1.0	5.0	5.1	5.2	4.4	3.2	3.2	2.0	2.0
2008	1.0	5.0	5.4	5.2	4.6	3.2	3.4	2.0	2.0
2009	1.0	5.0	5.7	5.2	4.8	3.2	3.5	2.0	2.0
2010	1.0	5.0	6.1	5.2	5.0	3.2	3.7	2.0	2.0
2011	1.0	5.0	6.4	5.2	5.1	3.2	3.8	2.0	2.0
2012	1.0	5.0	6.7	5.2	5.3	3.2	4.0	2.0	2.0
2013	1.0	5.0	7.1	5.2	5.5	3.2	4.1	2.0	2.0
2014	1.0	5.0	4.5	5.2	5.7	3.2	4.2	2.0	2.0
2015	1.0	5.0	4.7	5.2	5.9	3.2	4.4	2.0	2.0
2016	1.0	5.0	4.8	5.2	6.1	3.2	4.5	2.0	2.0
2017	1.0	5.0	5.0	5.2	6.3	3.2	4.7	2.0	2.0
2018	1.0	5.0	5.1	5.2	6.5	3.2	4.8	2.0	2.0
2019	1.0	5.0	5.3	5.2	6.7	3.2	5.0	2.0	2.0
2020	1.0	5.0	5.4	5.2	6.9	3.2	5.1	2.0	2.0

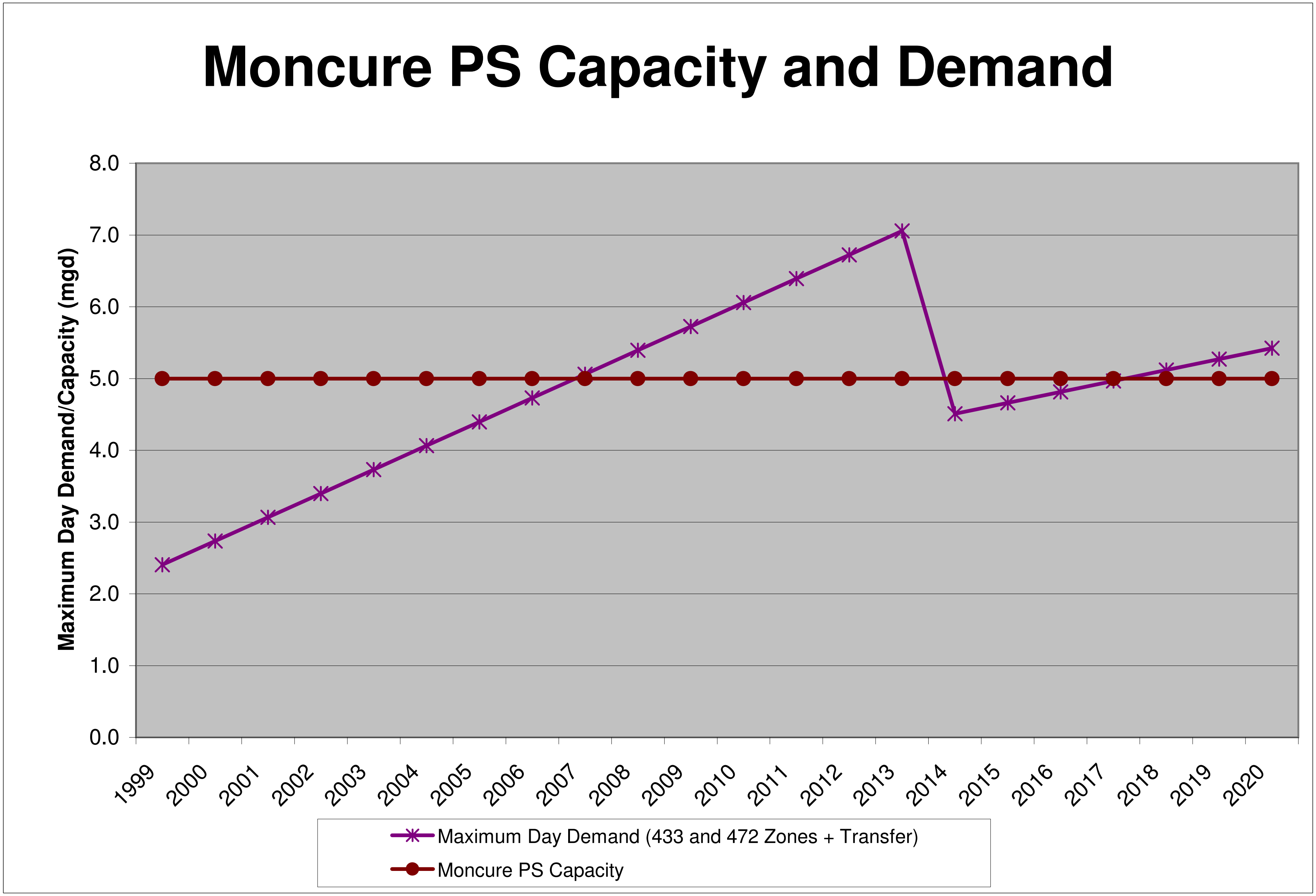
Smith Lake WTP Capacity and Demand



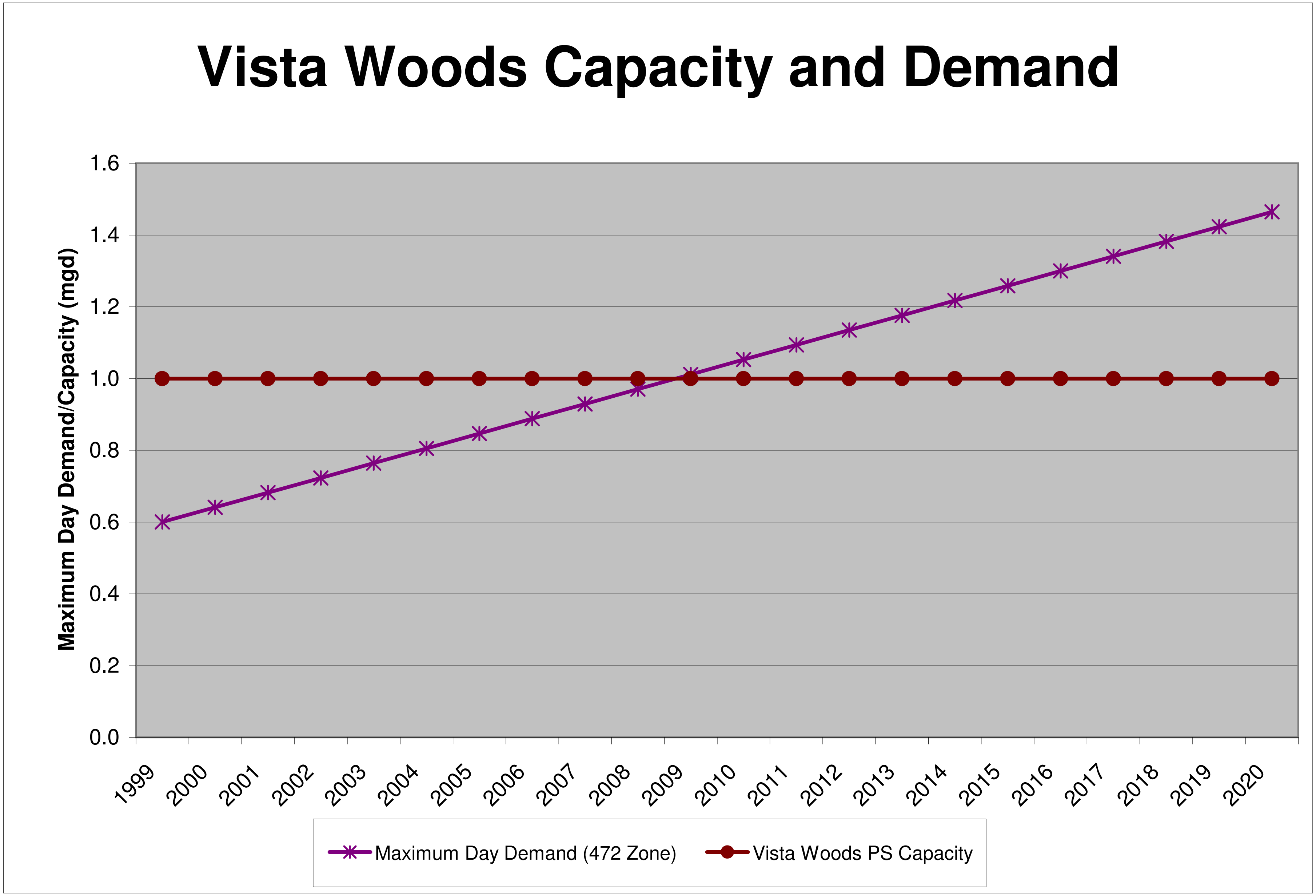
Smith Lake WTP actual capacity 10 mgd nominal, 15 mgd permitted, and 14

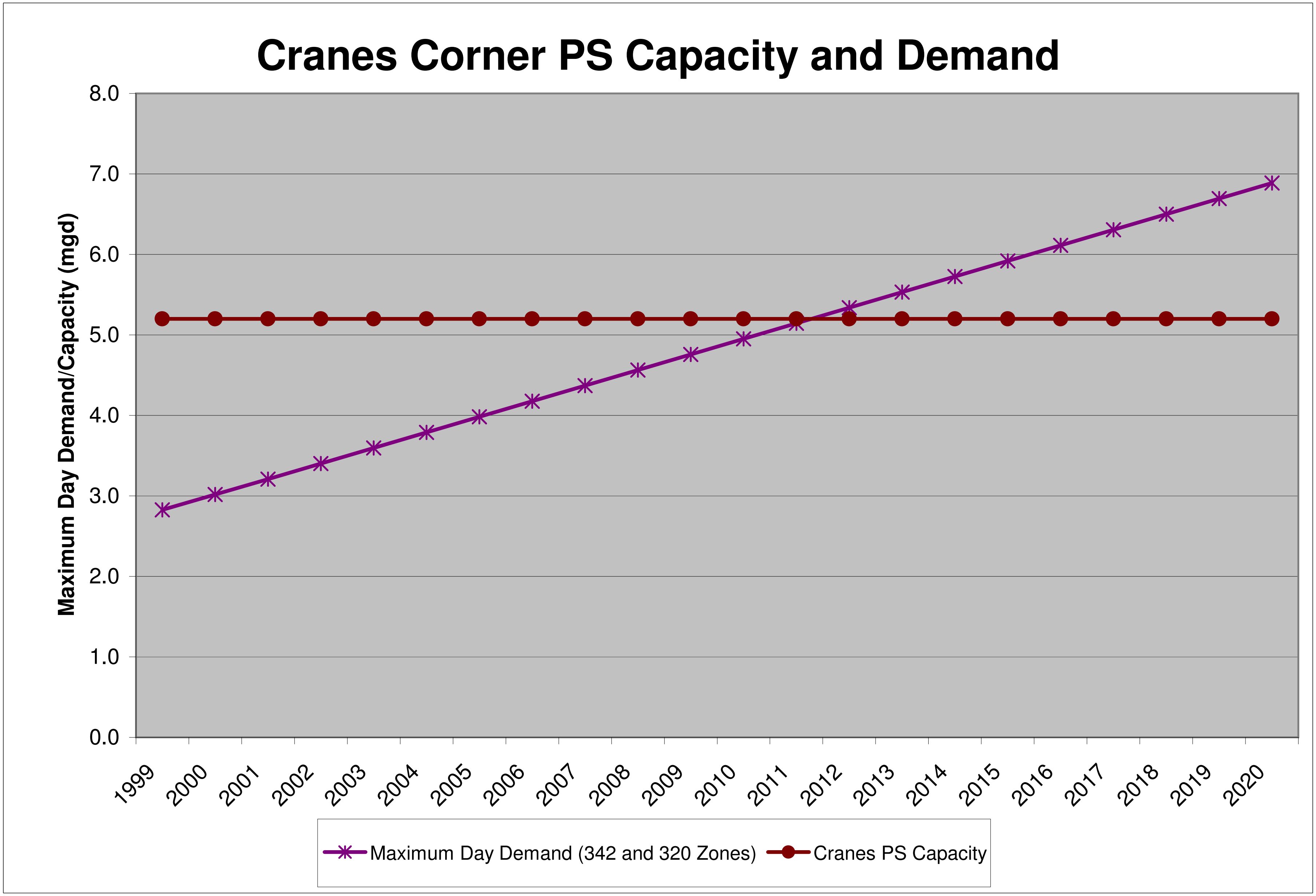


Abel Lake WTP Capacity and Demand
Maximum Day Demand/Capacity (mgd)
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
1999
2001
2003
2005
2007
2009
2011
2013
2015
2017
2019
Maximum Day Demand (342, 320, 370S, 503 Zones)
Abel Lake WTP Capacity
Maximum Day Demand (320, 370S, 503 Zones)

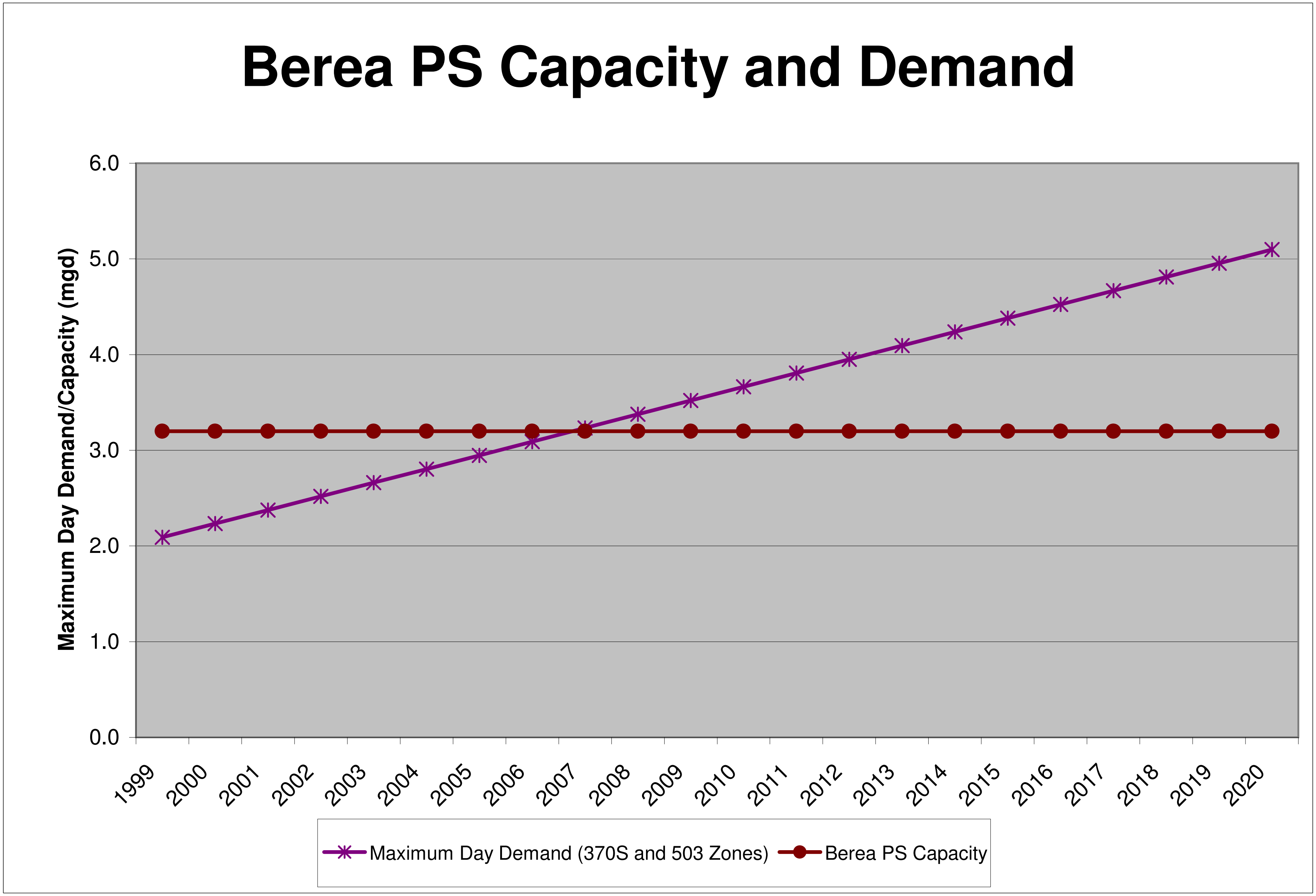


Moncure PS Capacity and Demand
Maximum Day Demand/Capacity (mgd)
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
1999
2000
2001
2002
2003
2004
2005
2006
2007
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
2018
2019
2020
Maximum Day Demand (433 and 472 Zones + Transfer)
Moncure PS Capacity

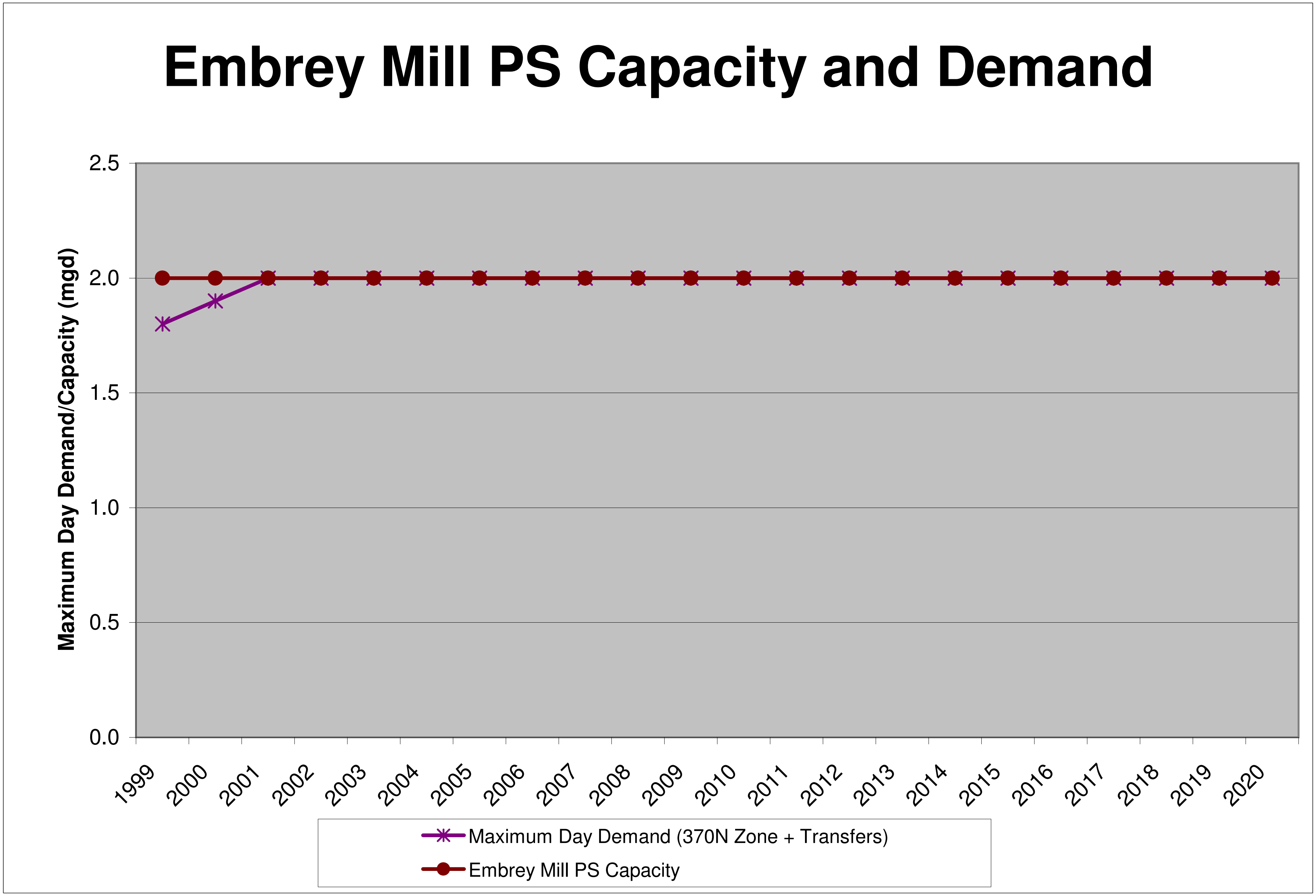

Vista Woods Capacity and Demand
Maximum Day Demand/Capacity (mgd)
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
1999
2000
2001
2002
2003
2004
2005
2006
2007
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
2018
2019
2020
Maximum Day Demand (472 Zone)
Vista Woods PS Capacity



Cranes Corner PS Capacity and Demand
Maximum Day Demand/Capacity (mgd)
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
1999
2000
2001
2002
2003
2004
2005
2006
2007
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
2018
2019
2020
Maximum Day Demand (342 and 320 Zones)
Cranes PS Capacity



Berea PS Capacity and Demand
Maximum Day Demand/Capacity (mgd)
6.0
5.0
4.0
3.0
2.0
1.0
0.0
1999
2000
2001
2002
2003
2004
2005
2006
2007
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
2018
2019
2020
Maximum Day Demand (370S and 503 Zones)
Berea PS Capacity


Embrey Mill PS Capacity and Demand
Maximum Day Demand/Capacity (mgd)
2.5
2.0
1.5
1.0
0.5
0.0
1999
2000
2001
2002
2003
2004
2005
2006
2007
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
2018
2019
2020
Maximum Day Demand (370N Zone + Transfers)
Embrey Mill PS Capacity

Appendix B

Pump Curves

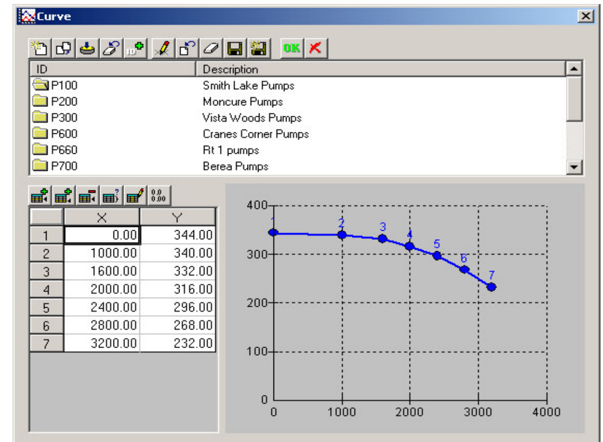
PUMP: ID (Char)	PUMP: DESCRIPT (Char)	PUMP: ZONE (Char)	OUTPUT: FLOW (gpm)	OUTPUT: HEADLOSS (ft)	PUMPHYD: CURVE (Char)
101	Smith Lake No. 1	310	2,842.83	264.15	P100
102	Smith Lake No. 2	310	2,842.83	264.15	P100
103	Smith Lake No. 3	310	0	0	P100
104	Smith Lake No. 4	310	0	0	P100
201	Moncure No. 1	433	1,418.65	165.17	P200
202	Moncure No. 2	433	1,418.65	165.17	P200
203	Moncure No. 3	433	0	0	P200
301	Vista Woods No. 1	472	608.16	40.79	P300
302	Vista Woods No. 2	472	0	0	P300
303	Vista Woods No. 3	472	0	0	P300
601	Cranes Corner No. 1	342	1,422.42	132.77	P600
602	Cranes Corner No. 2	342	1,422.42	132.77	P600
603	Cranes Corner No. 3	342	0	0	P600
701	Berea No. 1	503	1,629.48	250.53	P700
702	Berea No. 2	503	0	0	P700

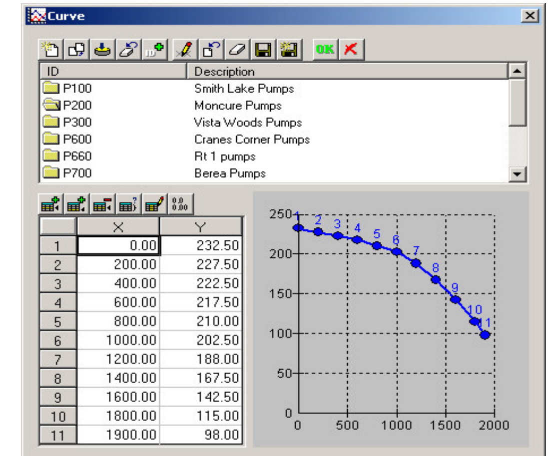
Notes:

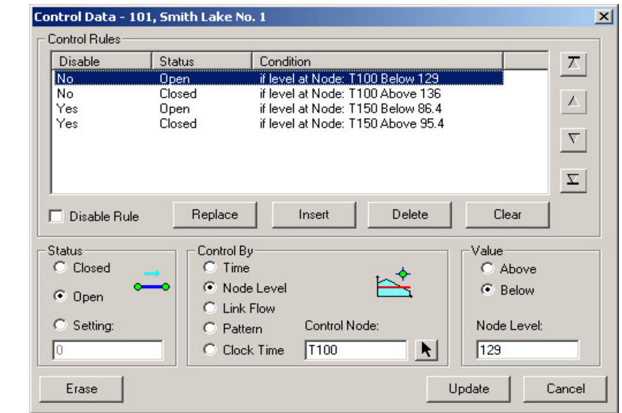
1. The following pumps are initially closed (turned off) in the model. They are turned on by their pump control settings.
   Pump 101 Smith Lake No. 1
   Pump 201 Moncure No. 1
   Pump 602 Cranes Corner No. 2

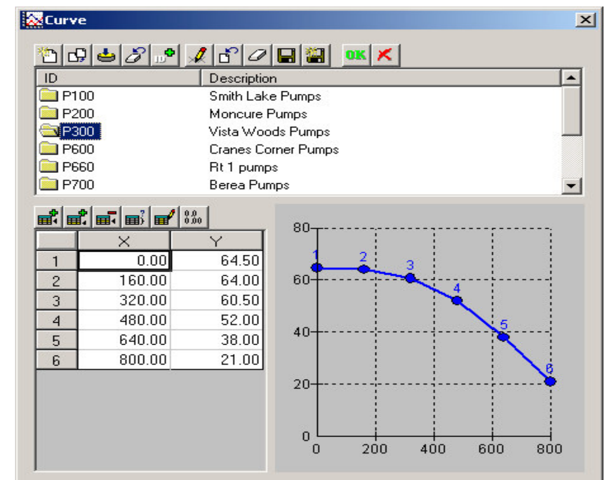
2. The following pumps are not inculded the present water system scenario (not assigned to a zone):
   661 M&M No.1
   680 Potomac Creek No. 1

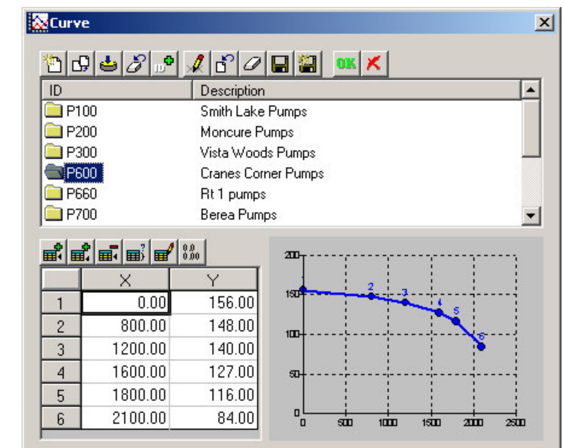
3. Pumps in Moncure Station were turned off in the model for Site 6.

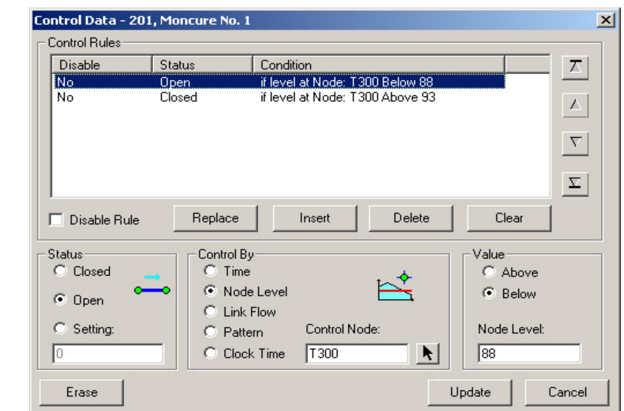


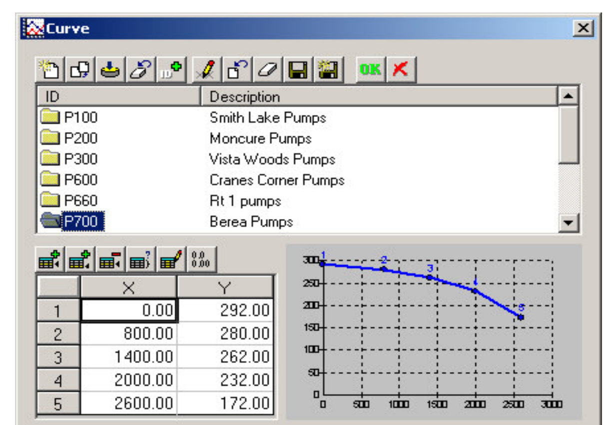


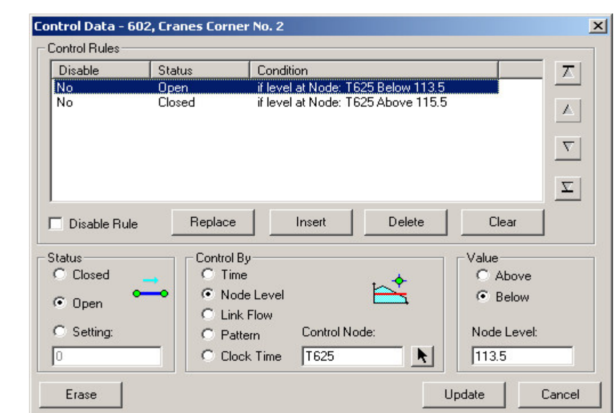






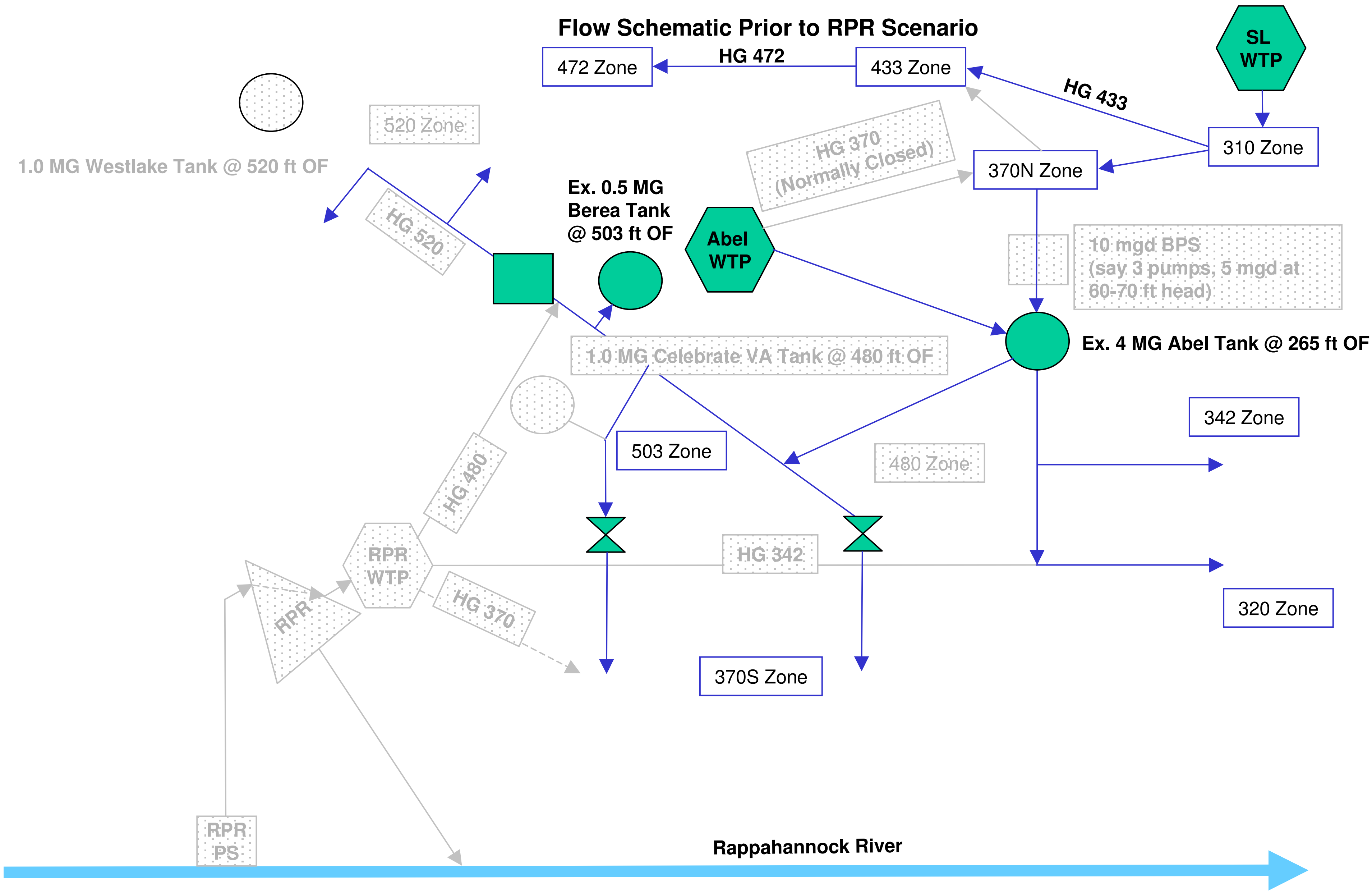




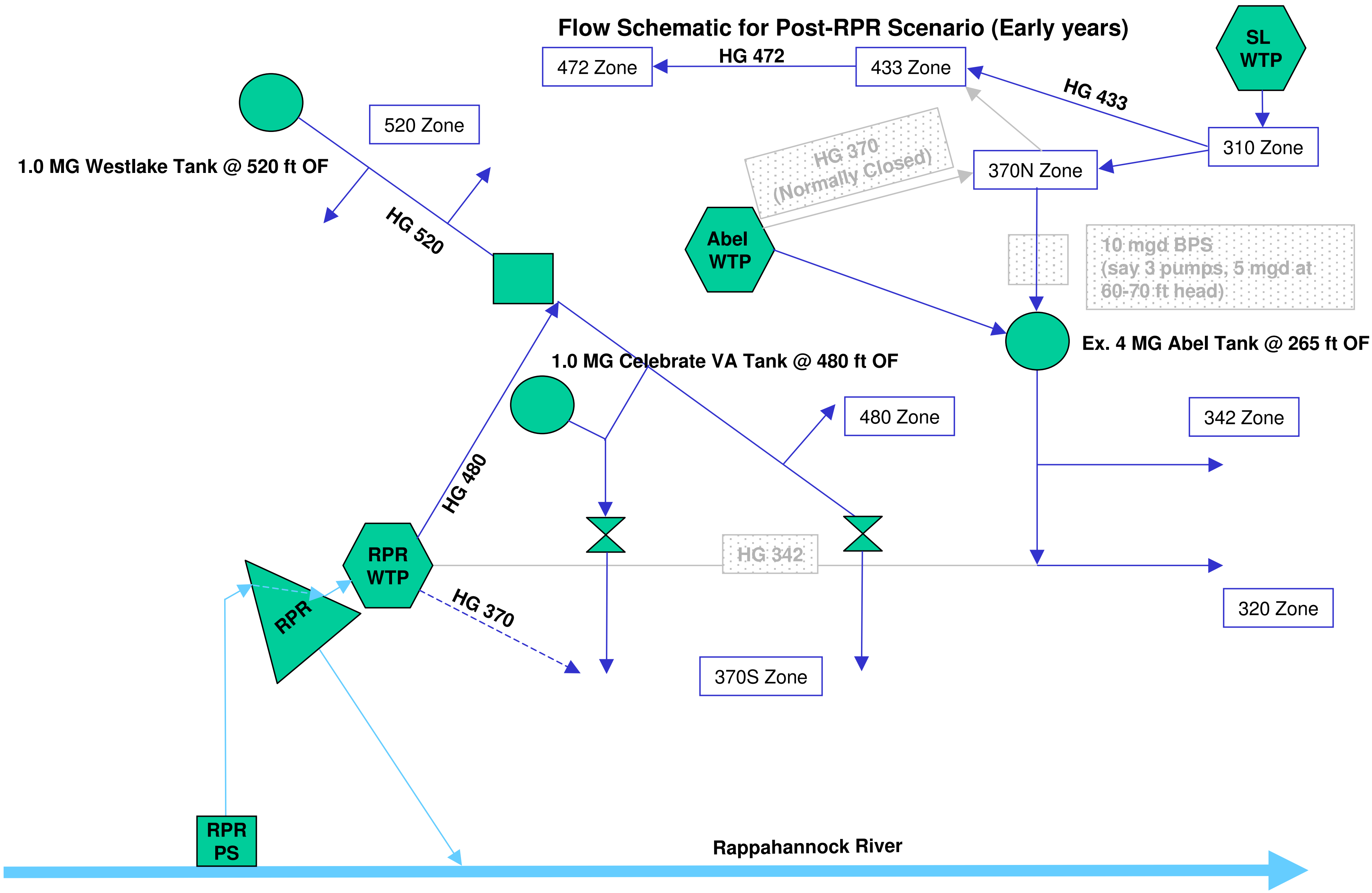


Appendix C

Water System Flow Schematics

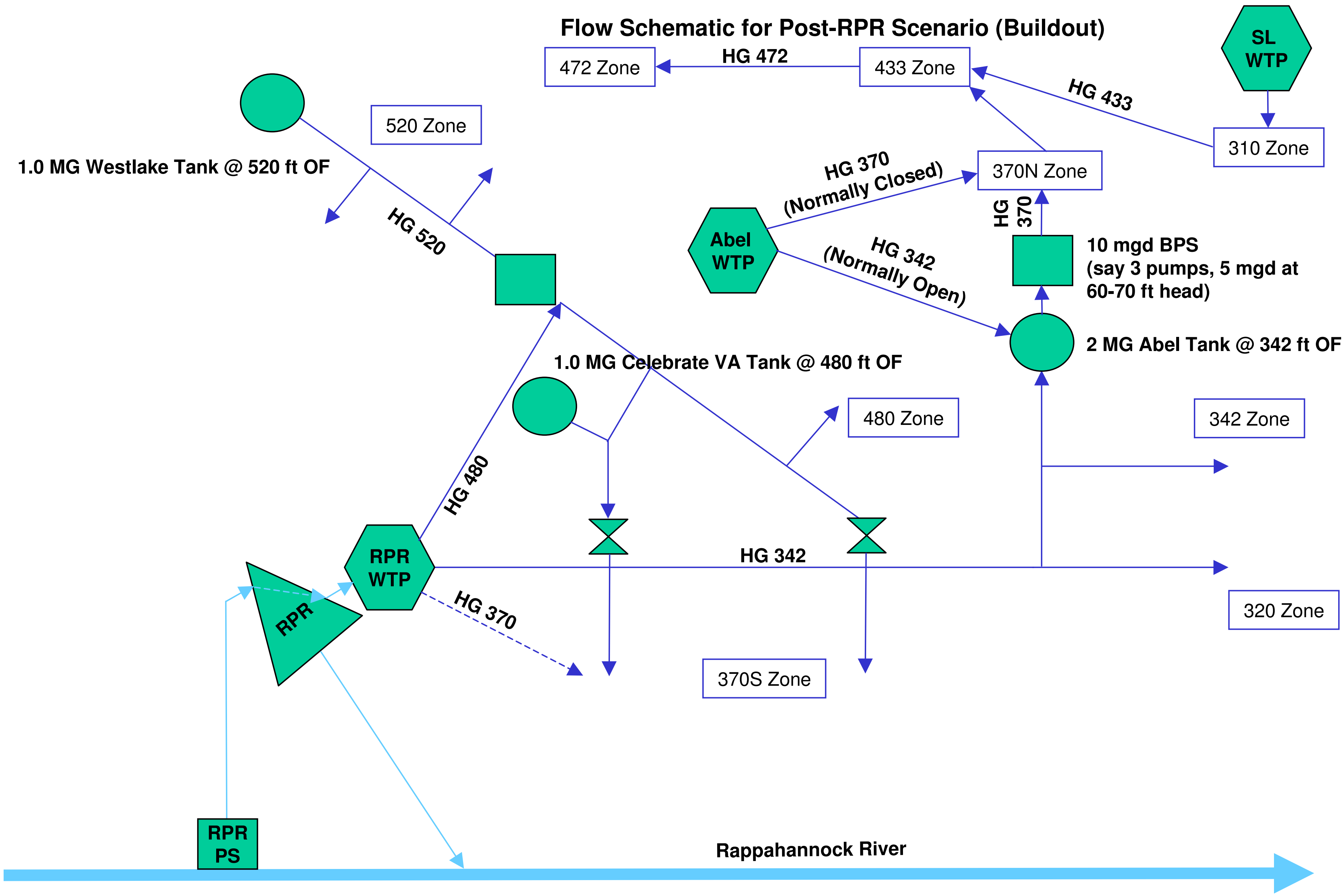


Flow Schematic Prior to RPR Scenario
472 Zone
HG 472
433 Zone
SL WTP
HG 433
310 Zone
520 Zone
1.0 MG Westlake Tank @ 520 ft OF
HG 370 (Normally Closed)
370N Zone
Ex. 0.5 MG Berea Tank @ 503 ft OF
HG 520
Abel WTP
10 mgd BPS (say 3 pumps, 5 mgd at 60-70 ft head)
Ex. 4 MG Abel Tank @ 265 ft OF
1.0 MG Celebrate VA Tank @ 480 ft OF
342 Zone
503 Zone
480 Zone
HG 480
RPR WTP
HG 342
RPR
HG 370
320 Zone
370S Zone
RPR PS
Rappahannock River



Flow Schematic for Post-RPR Scenario (Early years)
472 Zone
HG 472
433 Zone
HG 433
SL
WTP
310 Zone
520 Zone
1.0 MG Westlake Tank @ 520 ft OF
HG 520
HG 370
(Normally Closed)
370N Zone
Abel
WTP
10 mgd BPS
(say 3 pumps, 5 mgd at
60-70 ft head)
Ex. 4 MG Abel Tank @ 265 ft OF
1.0 MG Celebrate VA Tank @ 480 ft OF
480 Zone
342 Zone
HG 480
RPR
WTP
HG 342
320 Zone
RPR
HG 370
370S Zone
RPR
PS
Rappahannock River

Flow Schematic for Post-RPR Scenario (Buildout)



TECHNICAL MEMORANDUM 6

Rainfall/Flow Monitoring Program

Prepared for:	Stafford County Department of Utilities
Prepare by:	O'Brien & Gere
Date:	November 2004

This technical memorandum is one of a series being prepared for the Water and Sewer Master Plan project. The purpose of this technical memorandum includes:

- Summarizing the results of the rainfall data and flow data obtained from DOU's SCADA system for several pumping stations for the period from August 22, 2002 through September 25, 2002.
- Summarizing the results of the rainfall and flow monitoring program obtained during the I/I study for Austin/Whitson's Run for the period from March 26, 2003 through April 29, 2003.
- Estimating the amount of groundwater infiltration (GWI) and rainfall-dependent inflow and infiltration (RDI/I) into the DOU sewer system using the SCADA and flow monitoring data.

This document contains the following sections:

Terminology, Definitions and Glossary

Average Dry Weather Flow (ADWF) **–** ADWF consists of average daily sewage flows and groundwater infiltration (GWI). ADWF is the average flow that occurs on a daily basis with no evident reaction to rainfall.

C-factor **–** A measure of the interior roughness of a pipe**.**

Diurnal Demand or Flow – Fluctuation of water demands or wastewater flows over a 24-hour period.

Effective Storage – Effective storage for each storage facility is determined by establishing the level in each tank above which all points in the water system can be served at 20 psi or higher (based on peak hour or maximum day plus fire flow).

Equalization Storage **–** The storage of peaking flows to prevent overflows from the sewer collection and conveyance systems.

Groundwater Infiltration (GWI) **–** Groundwater that infiltrates pipeline and manhole defects located below the ground surface. Groundwater infiltration is separate and distinguished from inflow resulting from storm events. Infiltration is a steady 24-hour flow that usually varies during the year in relation to the groundwater levels above the sewers. Infiltration rates are normally estimated from wastewater flows measured in the sewers during the early morning hours when water use is at a minimum and the flow is essentially infiltration.

H2OMAP Water **–** H2OMAP Water is a computer model used for modeling the Department of Utilities' water system under various demand conditions.

H2OMAP Sewer **–** H2OMAP Sewer is a computer model used for modeling the Department of Utilities' sewer system under various flow conditions.

Inflow – Drainage that enters the collection system through illegal or permitted connections, such as catch basins, downspouts, area drains and manhole covers. Inflow is separate and distinguished from infiltration. The inflow rate can be determined from the flow hydrographs recorded with flow meters by subtracting the normal dry weather flow and the infiltration from the measured flowrate.

Infiltration/Inflow (I/I) **–** The wastewater component caused by rainfall-dependent infiltration/inflow (RDI/I) and groundwater infiltration (GWI).

Maximum Day Demand **–** The one day in the year when the consumption is the highest.

Maximum Hour Demand **–** The one hour in the year when water consumption is the highest.

Node – A junction of two or more pipes, commonly representing a point where pipe characteristics change.

Peak Dry Weather Flow (PDWF) – PDWF consists of peak sewage flows plus GWI. PDWF is the highest measured hourly flow that occurs on a dry weather day.

Peak Wet Weather Flow (PWWF) – PWWF consists of ADWF plus RDI/I. PWWF is the highest measured hourly flow that occurs during wet weather.

Peak Factor – Peak factor is PWWF/ADWF.

Pressure Reducing Valve (PRV) – A valve that will maintain a specified downstream pressure.

Pressure Zone – A network of water pipes having a common static hydraulic grade line. Pressure zones are separated by closed valves, pressure regulating valves, pumping stations, and reservoirs.

Rainfall-Dependent Infiltration/Inflow (RDI/I) – RDI/I consists of rainfall that enters the collection system through direct connections (roof leaders, manholes, etc.) and causes an almost immediate increase in wastewater flow.

Service Area – The area served by the water distribution or wastewater collection system.

Steady State Simulation – A network model solution for a single point in time.

Tributary Area – The tributary area of a sewage system consists of all areas that contribute flow to the sewer by gravity and/or force main discharges.

ADD	Average Day Demand
ADWF	Average Dry Weather Flow
AWWA	American Water Works Association
CIP	Capital Improvement Program
cfs	Cubic Feet per Second
CMOM	Capacity, Management, Operation and Maintenance
CWA	Clean Water Act
DOU	Stafford County Department of Utilities
D/DBP	Disinfectants/Disinfection Byproducts
EA	Environmental Assessment
EIS	Environmental Impact Statement
EPA	US Environmental Protection Agency
EPS	Extended Period Simulation
ft	Feet
FY	Fiscal Year
gpcpd	Gallons per Capita per Day
gpd	Gallons per Day
gpm	Gallons per Minute
gpdidm	Gallons per Day per Inch Diameter – Mile
GWI	Groundwater Infiltration
HAAs	Haloacetic Acids
HGL	Hydraulic Grade Line
ICR	Information Collection Rule
I/I	Infiltration and Inflow
IESWTR	Interim Enhanced Surface Water Treatment Rule
ISO	Insurance Service Organization
L	Liter
MCL	Maximum Contaminant Level
MDD	Maximum Day Demand

MG	Million Gallons
MGD	Million Gallons Per Day
mg/l	Milligrams per Liter
mgd	Million Gallons per Day
MPN/100 ml	Most Probable Number per 100 Milliliters
NEPA	National Environmental Policy Act
O&M	Operations and Maintenance
PDWF	Peak Dry Weather Flow
PHD	Peak Hour Demand
PRV	Pressure Reducing Valve
psi	Pounds per Square Inch
PSV	Pressure Sustaining Valve
PWWF	Peak Wet Weather Flow
PWS	Public Water Supply
RDI/I	Rainfall-Dependent Infiltration/Inflow
SCADA	Supervisory Control and Data Acquisition
SDWA	Safe Drinking Water Act
SSO	Sanitary Sewer Overflows
SWTR	Surface Water Treatment Rule
TCR	Total Coliform Rule
THMs	Trihalomethanes
UFW	Unaccounted-for Water
ug/L	Micrograms per Liter
USACE	US Army Corps of Engineers
USEPA	US Environmental Protection Agency
USGS	US Geological Survey
VDEQ	Virginia Department of Environmental Quality
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant

Executive Summary

Rainfall data and sewer flow monitoring data from the DOU's wastewater conveyance system were needed to calibrate the H2OMAP Sewer model and identify the system's response to storm events of varying characteristics. Rainfall and sewer flow data used in this study were collected for two periods: March through April 2003 and August through September 2002. In March through April 2003, DOU installed ten temporary flow meters in the wastewater conveyance system for 35 days to gather flow data within Austin/Whitson's Run Basin as part of an inflow/infiltration study. Rain gage data from Aquia WWTP and Quantico Marine Corps Base were used in the analysis. The data from each of the ten flow meters and SCADA data from four pumping stations in the Austin/Whitson's Run basin were reviewed, analyzed and summarized under dry and wet weather conditions. In addition to the March through April 2003 flow monitoring data for Austin/Whitson's Run, flow data from the DOU's SCADA system were obtained for eight pumping stations for a 35-day period from August 22 through September 25, 2002.

Groundwater infiltration (GWI) rates were calculated using flow monitoring data from March through April 2003 and August through September 2002. Based on the March through April 2003 results, groundwater infiltration (GWI) rates of 601 to 2,445 gpdidm were exhibited for the Austin/Whitson's Run basin with an average GWI rate of 1,282 gpdidm. For the August through September 2002 data, GWI rates ranged from 222 to 1,478 gpdidm with the average GWI rate of 509 gpdidm. Based on these data and the monthly wastewater treatment plant flow data during the dry weather conditions that occurred between July 2000 and September 2002, a groundwater infiltration rate of approximately 500 gpdidm appears reasonable.

The data from the dry and wet weather periods were compared to understand the range of basin peaking factors that existed in the DOU's sewer system and to estimate the amount of rainfall-dependent inflow and infiltration (RDI/I) impacting the different areas of the conveyance system. Based on the August 28, 2002 storm event, SCADA data from eight pumping station locations throughout the sewer system showed peaking factors ranging from 2.6 to 3.7 (i.e., peak hourly flows were 2.6 to 3.7 times greater than the average daily dry weather flow). For this storm event, the weighted average peaking factor for the system was approximately 2.8.

The data collected indicated that the storms captured during the 2002 and 2003 flow monitoring period had return intervals of 2-years or less. The limited intensity and volume of storms captured will have an impact on the wet weather calibration of the sewer model. Since the storms captured are relatively small, the model may have difficulty simulating larger storm events with longer return intervals. One option is to conduct additional flow monitoring to capture larger storms that can be used to provide additional calibration points.

Rainfall-dependent inflow and infiltration (RDI/I) rates were calculated for each of the flow monitoring locations for the August 28, 2002 storm event. Based on the results of the August 28, 2002 storm event, peak RDI/I rates of 0.31 to 4.47 gpm/manhole were exhibited for the eight pumping stations with 56 percent of the manholes in the sewer system exhibiting a weighted average peak RDI/I rate of 0.74 gpm/manhole (1,065 gpd/manhole).

1.0. Rainfall / Flow Monitoring Program

Flow and rainfall monitoring is used to quantify wastewater production (sanitary base flow) and rainfall-dependent I/I for the collection system. The data collected can be used for hydraulic evaluation of the sewer system, calibration of hydraulic models, assessment of I/I and effectiveness of rehabilitation measures in eliminating I/I.

In areas experiencing high flows during wet weather, an important parameter for proper interpretation and extrapolation from the flow data is the gathering of rainfall data. Achieving adequate density in the deployment of the rain gages to capture the variation in the size and intensity of storm cells is critical in developing reasonable relationships between rain and peak wastewater flows. Average rainfall for the study area can be developed using spatial relationships for individual rain gage locations. Increased accuracy is achieved through greater frequency and density in establishing rain gage sites. Alternatively, rainfall data may be captured and incorporated into the flow analysis through a combination of ground-based rain gages and Doppler radar rainfall data gathered by Next Generation Weather Radar Systems (NEXRAD) established at all major United States airports.

In March through April 2003, DOU installed ten temporary flow meters in the wastewater conveyance system for 35 days to gather flow data within Austin/Whitson's Run Basin as part of an I/I study. The meters were located throughout the Austin Run sewer system upstream of the pumping stations to allow isolation of flow basins. The data from each of the ten flow meters and SCADA data from four pumping stations in the Austin/Whitson's Run basin were reviewed, analyzed and summarized under dry and wet weather conditions.

In addition to the March through April 2003 flow monitoring data for Austin/Whitson's Run, flow data from the DOU's SCADA system was obtained for eight pumping stations in the sewer system for a 35-day period from August 22 through September 25, 2002. These data were also reviewed and analyzed under dry and wet weather conditions. Figures showing the configuration of the sewer system pumping stations are presented at the end of this technical memorandum.

2.0. Rainfall Data

2.1. Rain Gages

Rainfall data from the rain gages at the Aquia WWTP and Quantico Marine Corps Base were used to estimate the intensity, duration and volume of the storm events that occurred during the flow monitoring periods. Several storm events occurred during flow monitoring, including a significant event on August 28, 2002. The locations of several additional rainfall gages in Stafford and Fauquier Counties and rainfall totals for August 28, 2002 are shown in the figure at the end of this technical memorandum.

2.2. Storm Analysis

A number of storm events occurred during the flow monitoring and data collection periods. The rainfall data collected during the flow monitoring periods were used to estimate the return periods of the storms. Table 1 shows the return periods and the corresponding volumes for 24-hour storms in the DOU service area (Source: NOAA Atlas 14, National Weather Service).

Table 1: Return periods and corresponding storm characteristics for Stafford County

Return Period (years)	Volume for 24-hour event (inches)
100	8.12
50	6.94
25	5.90
10	4.71
5	3.94
2	3.05

An intensity-duration-frequency (IDF) curve can be used to estimate the return period for storm events of varying intensity and duration. The IDF curve for Richmond closely approximates the data presented in Table 1 for Stafford County. As shown on the Point Precipitation Frequency Estimates and the IDF curve at the end of this technical memorandum, it appears that the 3-inch, 24-hour storm event on August 28, 2002 was a 2-year storm event.

2.3. Summary of Rainfall and Storm Events

The rainfall and storm event data collected in this task will be an important part of the dry and wet weather hydraulic model calibration. The predicted RDI/I will be combined with the estimated dry weather wastewater flow and routed through the collection system using the hydraulic model for comparison with the field data collected.

Calibration of the hydraulic model is best when storms of varying characteristics are used. By using a set of storms with varying intensity, volume, and duration, RDI/I will be more accurately predicted. With the storm events captured during the flow monitoring periods, the return periods are fairly short so the hydraulic model can only be calibrated to these events. The characteristics of more severe events have to be extrapolated which can impact the predictive accuracy of the model.

3.0. Infiltration / Inflow Criteria

A benefit of collecting flow data for incorporation into the master planning component of the Capacity Assurance Planning (CAP) is the preliminary assessment of whether subsequent condition assessment work is needed through a SSES. Historical wastewater flow, gathered either at the wastewater treatment plant or at key subsystem locations, provides a preliminary basis for determining whether infiltration or inflow is excessive.

The October 1991 EPA handbook Sewer System Infrastructure Analysis and Rehabilitation is one resource available for determining whether the level of I/I in a collection system is excessive. The 1991 EPA criteria define the non-excessive infiltration as a flow rate that does not significantly exceed 120 gallons per capita per day (gpcpd). The sum of the domestic base flow and infiltration based on a 7-14 day average during high groundwater conditions is used as a basis of comparison when applying the EPA criteria. This assessment uses readily available information that an agency can assemble from existing sources.

These criteria were prepared over a decade ago and were used primarily in administering the grants program of that era. However, the current EPA discussions involving the “blending” rule for wet weather treatment plant operation again raised the issue of what constitutes excessive I/I. This 1991 handbook was again cited in the 2004 EPA documentation as the best available guidance for the preliminary evaluation of whether a collection system is subject to cost-effective I/I to pursue.

The value of these I/I criteria is that data are generally available. Population tributary to a control meter or a treatment plant, measured wastewater flows from within the collection system or at the plant, and a high groundwater reference provide the three basic elements for the analysis. A possible limitation of these per capita measures is that population densities often have little to do with leakage characteristics of the pipe. Sewer pipes leak due to external loading, weakening or deteriorating pipe and manhole materials, ineffective joint compounds, poor trench and bedding conditions, and the presence of groundwater or rain water, not as a result of tributary population.

To overcome this drawback, another measure developed during the period of the I/I federal grants program may be more useful. Leakage quantified as gallons per day per inch-diameter of sewer (gpdidm) measures were routinely utilized to qualify individual sewer segments initially and later entire subsystems or sewersheds for further infiltration evaluation. The inch-diameter miles of a collection system are derived from the length of sewer expressed in miles times the diameter in inches. The computation is typically performed incrementally by each sewer segment or estimated by multiplying an average diameter across the length of the entire collection system in the study.

These inch-diameter mile measures were useful because they incorporated properties or measures of the leakage expressed by the physical characteristics (length and diameter) of the pipe. The rates also provide standardized units for ranking or prioritizing subsystems for subsequent study, independent of the differences in the size or lengths of the pipe in the actual subsystems. Table 2 presents a series of these non-excessive criteria devised by the EPA over the years of the grant program.

Table 2: Selected historical excessive infiltration / inflow criteria

Criteria Source	Criteria for Non-excessive Infiltration / Inflow Determination
EPA Program Requirements Memorandum (PRM 78-10, 1978)	Established 1500 gpdidm as non-excessive leakage allowance, perform a cost-effective analysis to determine if the leakage is possibly excessive and qualifies for investigation.
Draft Program Requirements (PRM 80, 1980)	Proposed 3000 gpdidm as non-excessive allowance, maximum of 30% infiltration removal for use in cost-effective analysis.
EPA Handbook: Procedures for Investigating Infiltration / Inflow, (EPA 68-01-4913, 1981)	Non- Excessive Allowance Ranges 2,000 – 3,000 gpdidm for sewer lengths greater than 100,000 lf 3,000 – 5,000 gpdidm for sewer lengths between 50,000 and 100,000 lf 5,000 – 8,000 gpdidm for sewer lengths between 1,000 and 50,000 lf
EPA Handbook: Facilities Planning, 1981	Non- Excessive Allowance Ranges 2,000 – 3,000 gpdidm for sewer lengths greater than 100,000 lf 3,000 – 6,000 gpdidm for sewer lengths between 10,000 and 100,000 lf 6,000 – 10,000 gpdidm for sewer lengths less than 10,000 lf
EPA Handbook: Sewer System Infrastructure Analysis and Rehabilitation (EPA 625/6-91/030, 1991)	Non-Excessive Infiltration Preceding year's 7-14 day high ground water wastewater flow less than 120 gpcpd.

Criteria Source	Criteria for Non-excessive Infiltration / Inflow Determination
	Non-Excessive Inflow Total daily average storm flow less than 275 gpcpd. No operational problems in collection system and WWTP.

Both the 1991 handbook criteria and the inch-diameter mile criteria have also been developed as threshold criteria for the inflow assessment (Table 2). Cumulative and subsystem inflow rates should be determined for each subsystem to provide a spatial distribution of inflow. In systems with significant inflow, a comparison of cumulative inflow and subsystem-generated inflow rates should show that the cumulative inflow for interior subsystems is less than the sum of the individual subsystem-generated inflows. This would be consistent with expected system flow dynamics in which peak flows are dampened as they travel through the system.

Prior to this Master Plan, DOU used the following infiltration allowances for planning purposes:

- 100 gpdidm for proposed sewers and sewers less than 10 years old.
- 1000 gpdidm for sewers 10-20 years old.
- 2000 gpdidm for sewers > 20 years old
- Based on flow monitoring data if available.

Comparing the information in Table 2 with the DOU criteria, it appears that the DOU infiltration allowances are on the lower end of the range. However, using 3,477 inch-diameter miles for the sewer system in the H2OMAP Sewer model and an infiltration allowance of 2,000 gpdidm yields a total infiltration of approximately 7 mgd which exceeds the existing average wastewater flow. Based on 500 gpdidm, the groundwater infiltration during the dry weather conditions that occurred in 2001 (no groundwater monitoring wells available) is estimated to be roughly 1.74 mgd. Subtracting 1.74 mgd from an average wastewater flow of 6 mgd in 2001 yields a sanitary base flow of approximately 4.26 mgd which is roughly 68 gpcpd of sanitary base flow (assuming a population of 63,000 served). This appears to be a reasonable estimate of the groundwater infiltration and correlates well with the flow monitoring data presented in the following section.

Prior to this Master Plan, DOU used the following inflow allowances for planning purposes:

- 500 gpd/manhole for proposed manholes and manholes less than 10 years old.
- 1500 gpd/manhole for manholes 10-20 years old.
- 2000 gpd/manhole for manholes > 20 years old
- Based on flow monitoring data if available.

A comparison of the quantity of inflow to the sewer system was computed using the DOU criteria and the information from the above table. Based on roughly 8,673 manholes in the sewer system and 2000 gpd/manhole (average age greater than 20 years old), the estimated inflow to the sewer system is approximately 17.3 mgd. Adding 6.5 mgd of base sanitary flow in 2002 to 17.3 mgd of inflow yields approximately 23.8 mgd of peak wet weather flow. From the above table, using a total daily average storm flow of 275 gpcpd and a population of 63,000 in the sewer service area yields an inflow of 17.3 mgd, the estimated peak wet weather flow is 23.8 mgd. The peaking factor (peak wet weather flow / average dry weather flow) for these calculations was 3.6. As described later in Technical Memorandum 8 (Wastewater Collection, Pumping and Conveyance Facilities), this peaking factor closely approximates the 3.5 peaking factor used in this Master Plan to estimate the peak wet weather flow.

4.0. Flow Data Analysis

The sewer model used to analyze DOU's sewer system improvements was calibrated for dry weather conditions and subsequently calibrated for wet weather conditions by adding RDI/I to the dry weather flow. The following sections describe how the components of the dry and wet weather flows were derived.

4.1. Dry Weather Flow Analysis

During dry weather conditions, flow in the sewer system is the sum of the sanitary base flow and groundwater infiltration (GWI). The dry weather flow at a point in the sewer system can be calculated as follows:

Average Dry Weather Flow (ADWF) = Sanitary Base Flow + Groundwater Infiltration (GWI)

Where:

Average Dry Weather Flow (ADWF) is the average flow that occurs in sanitary sewers on a daily basis with no evident reaction to rainfall.

Sanitary base flow used for model calibration is equal to 80% of the water meter billing data for 2001 which is an estimate of the customer water demand that is returned to the sewer.

Groundwater infiltration (GWI) is an allowance that is added to the sanitary base flow (derived from sewage flow factors) to obtain the dry weather flow. GWI represents flow that is separate and distinguished from inflow resulting from storm events during wet weather conditions.

4.1.1. Groundwater Infiltration (GWI)

The rainfall and flow data were analyzed to determine the components of the dry weather flow and to provide a starting point in the analysis of the wet weather flow data. For both the 2002 and 2003 flow monitoring data, three days of dry weather (i.e., no rainfall) were used in the analysis. The three days of dry weather flow were August 22-25, 2002 and April 16, 24, 28, 2003 (each of these days was preceded by a period of dry weather as shown in Figure 1 for the interceptor upstream of Basin 8).

For each flow meter, the amounts of sanitary base flow (derived from residential, commercial, and industrial uses) and groundwater infiltration (GWI) were determined. Wastewater flows in the early morning hours between 12:00 a.m. and 5:00 a.m. were assumed to be predominately groundwater infiltration (lowest flows shown on Figure 1). The GWI rate was determined by dividing the sewer flow at the location by the upstream inch-diameter miles of pipeline. Tables 3 and 4 present the results of the GWI analyses.

Figure 1: Stafford County - Austin Run Sewer Evaluation
Basin 08 (MH 40-0105)

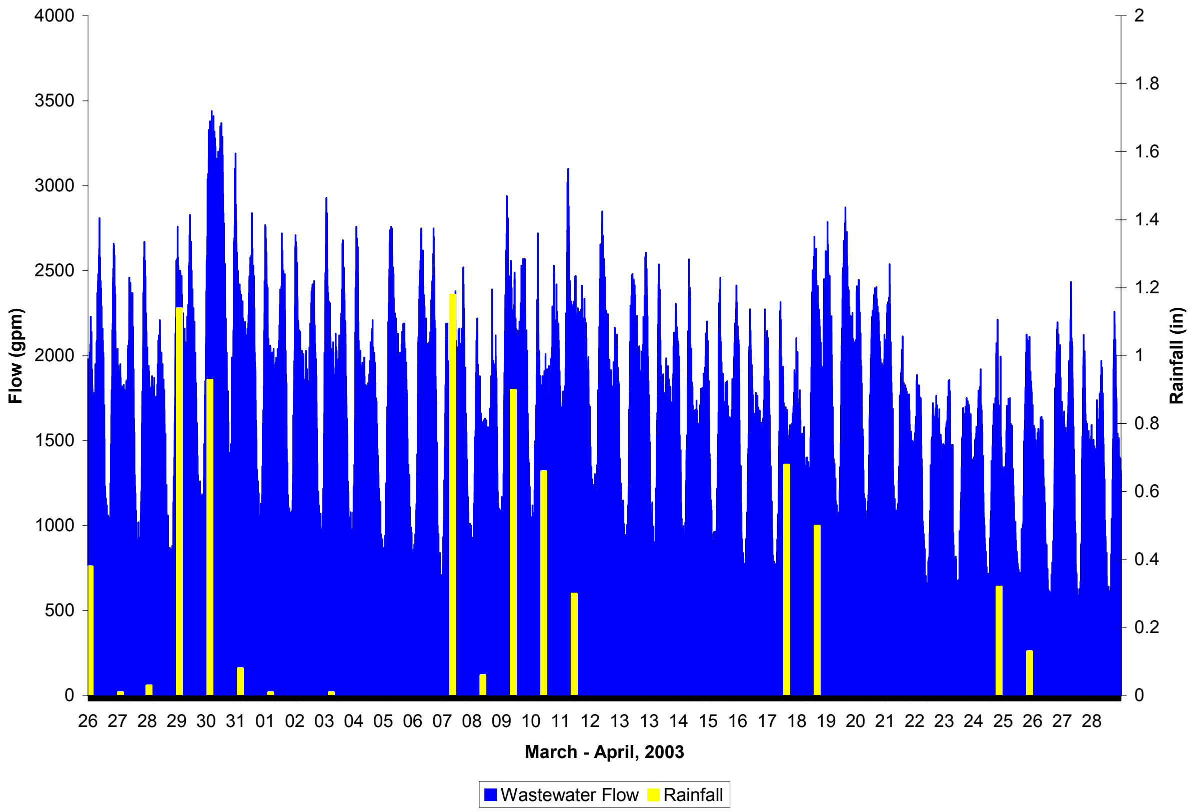


Table 3: Infiltration rates based on Austin Run/Whitson's Run flow monitoring data (March – April 2003)

Location		Flow Monitoring Period	Upstream Basins	Base Flow				Total Inch-Miles	Infiltration Rate (gpdidm)
Basin #	Manhole/ P.S. ID			Total (gpm)	Total (gpd)	Upstream Basins (gpd)	Net (gpd)		
Basin Infiltration									
5	40-0131	May - June	4	600	864,000	410,400	453,600	27.73	16,358
4	40-0138	May - June	3, 13, 14	285	410,400	250,560	159,840	65.37	2,445
9	40-0124	March - April	--	15	21,600	0	21,600	24.14	895
7	40-0116	May - June	6, 9	660	950,400	885,600	64,800	73.27	884
2	40-0214	March - April	1	107	154,080	89,280	64,800	80.51	805
8	40-0105	March - April	7, 10	900	1,296,000	1,274,400	21,600	35.40	610
1 [1]	40-0225	March - April	11, 12	ND	ND	ND	ND	ND	ND
3 [2]	40-0206	ND	2	ND	ND	ND	ND	ND	ND
6 [3]	40-0125	May - June	5	ND	ND	ND	ND	ND	ND
10 [4]	40-2001	March - April	--	225	324,000	0	324,000	213.50	1,518
Pump Station Infiltration									
14 [5]	PS 19	NA	--	52	74,880	0	74,880	0.23	325,565
11 [6]	PS 51	NA	--	27	38,880	0	38,880	26.23	1,482
13 [7]	PS 38	NA	--	15	21,600	0	21,600	23.18	932
12 [8]	PS 47	NA	--	35	50,400	0	50,400	57.21	881
Total System Inch-Miles (Basin 1 – Basin 14)									**626.77**
Total System Base Flow - Basin 8 (gpd)									**1,296,000**
Infiltration Rate for Austin Run/Whitson's Run area (gpdidm)									**2,068**
System Inch-Miles for all basins excluding Basins 5 and 14									**598.81**
System Base Flow for all basins excluding Basins 5 and 14 (gpd)									**767,520**
Infiltration Rate for Austin Run/Whitson's Run area excluding Basins 5 and 14 (gpdidm)									**1,282**

Notes:

1. Adequate data were not obtained for Basin 1, as the flow meter stopped working after approximately 3 days of metering. The infiltration analysis was performed as if Basin 2 encompassed Basin 1 as well. Total inch-miles for the Basin 2 analysis, therefore, include Basins 1 and 2 and pump stations upstream of both basins.
2. Adequate data were not obtained for Basin 3, as the flow meter did not work during the second metering period. Data obtained during the first metering period were not accurate. The infiltration analysis was performed as if Basin 4 encompassed Basin 3 as well. Total inch-miles for the Basin 4 analysis, therefore, include Basins 3 and 4 and pump stations upstream of both basins.
3. Base flow information for Basin 6 was inadequate as flows "bottomed out" during low flow conditions. The infiltration analysis was performed as if Basin 7 encompassed Basin 6 as well. Total inch-miles for the Basin 7 infiltration analysis, therefore, include Basins 6 and 7.
4. Although there were pump stations upstream of Basin 10 they were not utilized, as the intent for metering Basin 10 was to provide an overall Base Flow Rate for the Austin Run Interceptor. By obtaining this base flow rate, the infiltration rates for Whitson's Run could be isolated by subtracting out Austin Run flows. The Basin 8 infiltration analysis reflects this by subtracting out Basin 10 flows. A detailed analysis, including an infiltration analysis, is being performed on the Austin Run Interceptor during Phase II.
5. Basin 14 includes flow from the collection system tributary to PS 19; actual flow data were collected at PS 19.
6. Basin 11 includes flow from the collection systems tributary to PS 51 and PS 52; actual flow data were collected at PS 51.
7. Basin 13 includes flow from the collection systems tributary to PS 37 and PS 38; actual flow data were collected at PS 38.
8. Basin 12 includes flow from the collection systems tributary to PS 47, PS 54 and PS 55; actual flow data were collected at PS 47.

Table 4: Infiltration rates based on August – September 2002 flow monitoring data

Pump Station Identification			Upstream Pumping Stations	Inch-Miles			Infiltration		
GIS PS #	**SCADA PS #**	**Location**		**Pumping Station**	**Upstream PS**	**Total Inch-Miles**	**(gpm)**	**(gpd)**	**(gpdidm)**
1	79	Aquia Harbour	2, 3, 4, 5, 6	3.39	29.81	33.2	5.11	7,358	222
2	47	Aquia Harbour	3, 4, 5, 6	9.00	20.81	29.81	4.91	7,070	237
5	80	Aquia Harbour	none	6.56	0	6.56	1.47	2,117	323
20	4	Aquia Harbour	1, 2, 3, 4, 5, 6, 21, 22, 24, 27, 28, 29, 30, 31, 32, 34, 35, 36	23.64	171.12	194.76	85.02	122,429	629
31	2	Aquia Harbour	1, 2, 3, 4, 5, 6, 32, 34, 35, 36	35.97	96.33	132.3	23.25	33,480	253
								Aquia Harbour Average Infiltration	**333**
40	6	Austin Run	19, 25, 26, 39, 41, 42, 43, 44, 45, 47, 48, 49, 50, 51, 52, 54, 55, 56, 58	718.52	440.11	1158.63	553.2	796,608	688
47	11	Austin Run	54, 55	46.59	10.62	57.21	11.44	16,474	288
49	20	Austin Run	none	160.18	0	160.18	164.37	236,693	1,478
58	44	Austin Run	none	103.94	0	103.94	19.95	28,728	276
								Austin Run Average Infiltration	**682**
60	18	Rappahannock	61, 62, 63, 64, 66	311.74	115.38	427.12	0.11	158	0
64	36	Rappahannock	none	37.31	0	37.31	15.83	22,795	611
80	10	Rappahannock	60, 61, 62, 63, 64, 65, 66, 67, 68, 70, 72, 75, 75A, 81, 82, 83, 84, 86, 89, 90, 90A, 91, 92, 93, 93A, 94, 95, 96, 97, 98, 99	452.77	813.27	1266.04	967.58	1,393,315	1,101
								Rappahannock Average Infiltration	**571**
								TOTAL SYSTEM AVERAGE INFILTRATION	**509**

Notes:
1. The above infiltration rates are based upon flow data provided to O'Brien & Gere by DOU personnel for various pump stations throughout the County.

Comparing the infiltration rates presented in Tables 3 and 4, it appears that the high infiltration rates that occurred during Spring 2003 were likely due to the prolonged periods of extreme wet weather. For this study, the groundwater infiltration (GWI) component of the dry weather flow is estimated to be 500 gpdidm.

Wastewater treatment plant flow data for August 2002 were used to check the reasonableness of the GWI rate. Using 3,477 inch diameter/mile for the overall sewer system (not including service connections) and 500 gpdidm for the GWI rate yields approximately 1.74 mgd of GWI. Subtracting 1.74 mgd from the August 2002 monthly average wastewater treatment plant flow of 6.16 mgd (assumed to be essentially dry weather flow due to drought) yields 4.42 mgd of sanitary base flow which equates to roughly 70 gpd per person (assuming approximately 63,000 customers). The calculated sanitary base flow and GWI rate appear to be reasonable estimates.

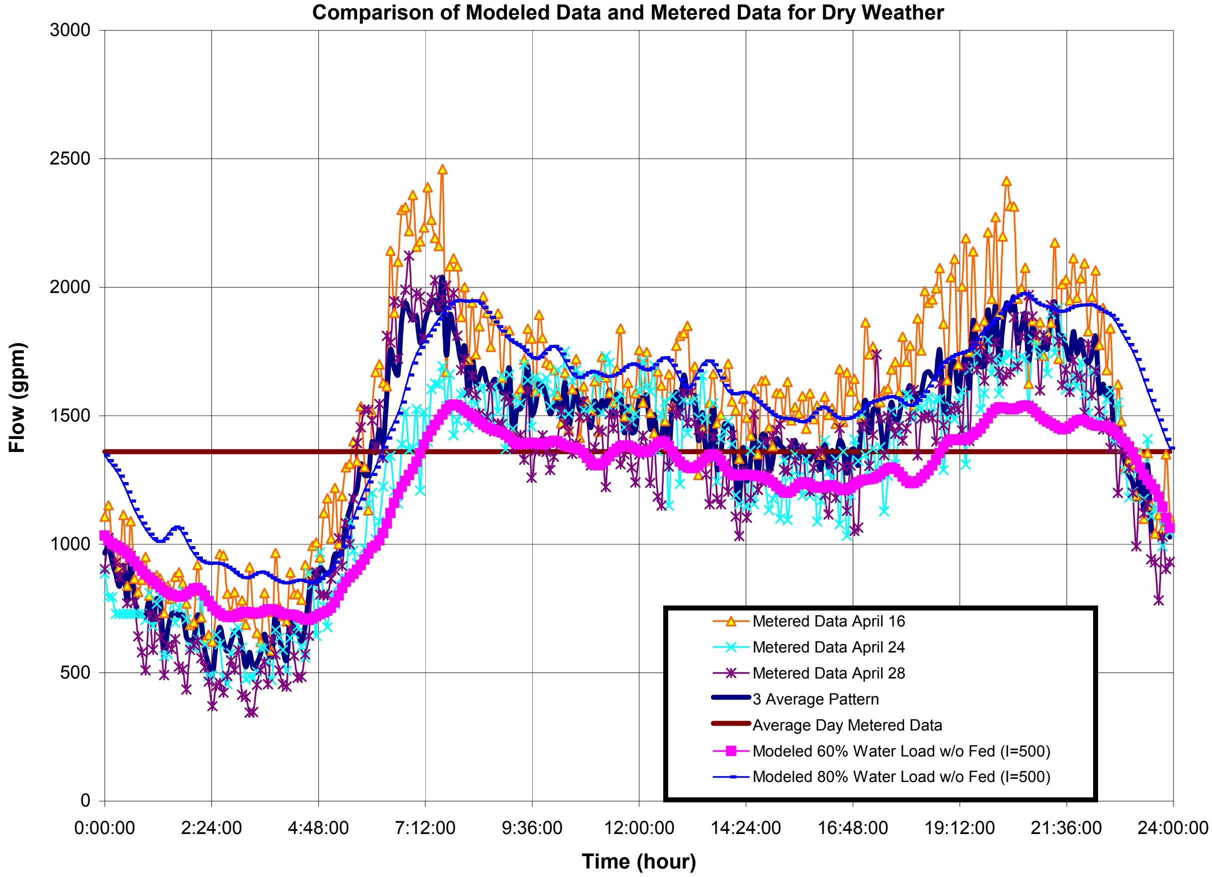
4.1.2. Dry Weather Flow Pattern

The average dry weather flow can be approximated by using either direct measurement of average daily dry weather flow during dry weather/low groundwater conditions or winter month water consumption data. The instantaneous average dry weather flow varies throughout each day, with the highest rates normally occurring between 8:00 a.m. and 11:00 a.m., depending on collection system size and characteristics. The ratio of peak 60-minute flow to total average daily flow is defined as the diurnal peaking factor.

Using the dry weather flow data, an average dry weather day for each location was estimated by averaging the hourly flow rates for those days. Figure 2 shows the dry weather flow pattern for a flow monitoring point near Basin 8. To assess the validity of the sanitary base flows and the GWI rates, data was input to the sewer model and is shown on Figure 2. For model calibration, sanitary base flows at 80% of water demand and 60% of water demand were evaluated. As shown on Figure 2, using 80% of the water demand as the return flow for the upstream sewer basin and a GWI rate of 500 gpdidm produces model flows that closely match the peaks and the pattern of the average flow measured for the three dry weather days in August 2002. Therefore, these dry weather components appear to be accurate estimates.

Figure 2: Gravity Main P40-0105A & Manhole 40-0105

Comparison of Modeled Data and Metered Data for Dry Weather



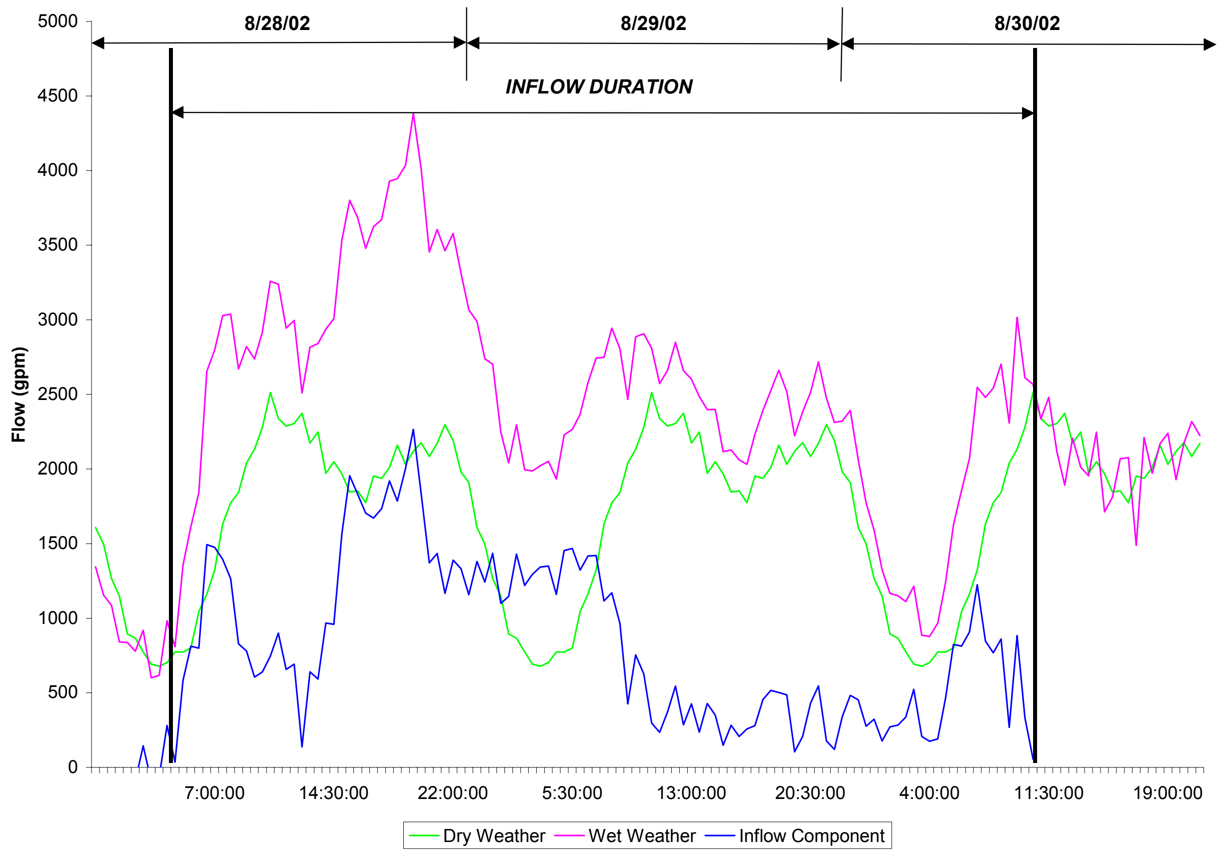
4.2. Wet Weather Flow Analysis

The formula for calculating the sewer loads for wet weather conditions is as follows:

Peak Wet Weather Flow (PWWF) = Average Dry Weather Flow (ADWF) + RDI/I

As discussed in the rainfall section above, the largest storm events that occurred during the 2002 flow monitoring period were evaluated. These storms generated the largest system response and were selected as the RDI/I interval. Once the average daily dry weather flow was determined for a monitoring location, the wet weather analysis could be conducted. The hourly dry weather flow data during the inflow period of the storm event were subtracted from the corresponding hourly wet weather flows to determine the additional flow input due to the storm event (i.e., rainfall-dependent I/I). Figure 3 shows the flow components under wet weather conditions for the Austin Run Pumping Station.

Figure 3: Inflow Hydrograph: PS # 40 (SCADA # 6)



A peaking factor was derived for each monitoring location by dividing the peak wet weather flow (PWWF) by the average dry weather flow (ADWF) for the day. Table 5 lists the average dry weather flows (ADWF) for the day, peak wet weather flows (PWWF) for the August 28, 2002 storm event, and the corresponding peaking factors at each location.

Table 5. Wet weather peaking factors for August 28, 2002 storm event

Pump Station ID			Number of Upstream Manholes	Avg. Daily Dry Weather Flow (gpm)	Peak Wet Weather Flow [1] (gpm)	Inflow Peaking Factor [2]
GIS #	SCADA #	Basin				
1	79	Aquia Harbour	minimal wet weather response			
2	47	Aquia Harbour	minimal wet weather response			
5	80	Aquia Harbour	minimal wet weather response			
20	4	Aquia Harbour	171	322	1,063	3.3
31	2	Aquia Harbour	220	174	643	3.7
40	6	Austin Run	2,514	1,717	4,383	2.6
47	11	Austin Run	202	54	161	3.0
49	20	Austin Run	461	413	1,150	2.8
58	44	Austin Run	224	51	131	2.6
60	18	Rappahannock	1,092	514	1,536	3.0
64	36	Rappahannock	142	67	250	3.7
80	10	Rappahannock	bad data during storm event			
Aquia Harbour Weighted Average Peaking Factor [3]						3.5
Austin Run Weighted Average Peaking Factor [3]						2.6
Rappahannock Weighted Average Peaking Factor [3]						3.1
Entire System Weighted Average Peaking Factor [3]						**2.8**

Notes:
1. Peak wet weather flow for all pumping stations was based on a 3-inch storm event that occurred on August 28, 2002.
2. The Inflow Peaking Factor was determined by dividing the peak wet weather flow by the average daily dry weather flow.
3. Number of upstream manholes was used as the weighting factor. Total manholes used to compute site-specific peaking factors for the locations in Table 5 is 5,026 (58% of 8,673 total manholes in sewer system).

Table 6 shows the RDI/I rates calculated for several locations in the sewer system based on the August 28, 2002 storm event.

Table 6. Peak RDI/I rates for August 28, 2002 storm event

Pumping Station	Location	Number of Upstream Manholes	Peak Inflow (gpm/manhole)	Peak Inflow (gpd/manhole)
PS # 20 (SCADA # 4)	Aquia Harbour	171	4.47	6,437
PS # 31 (SCADA # 2)	Aquia Harbour	220	1.12	1,613
PS # 40 (SCADA # 6)	Austin Run	2,514	0.67	965
PS # 47 (SCADA # 11)	Austin Run	202	0.50	720
PS # 49 (SCADA # 20)	Austin Run	461	1.12	1,613
PS # 58 (SCADA # 44)	Austin Run	224	0.31	446
PS # 60 (SCADA # 18)	Rappahannock	1,092	0.80	1,152
PS # 64 (SCADA # 36)	Rappahannock	142	0.88	1,267

Removing PS # 20 (SCADA # 4) as an outlier, approximately 56 percent of the manholes in DOU's sewer system (4,855 out of 8,673 total manholes) are upstream of the remaining pumping stations shown in Table 6. The weighted average peak RDI/I rate for these remaining pumping stations is 0.74 gpm/manhole which is approximately 1,065 gpd/manhole. For the existing system (8,673 manholes), the peak inflow generated from the August 28, 2002 storm event is approximately 9.2 mgd (1,065 gpd/manhole x 8,673 manholes). Adding the August 2002 monthly average wastewater treatment plant flow of 6.16 mgd (assumed to be essentially dry weather flow due to drought) to the 9.2 mgd of inflow from the August 28, 2002 storm event yields and estimated peak flow of approximately 15.4 mgd at the wastewater treatment plants. Actual flow data measurements at the wastewater treatment plants on August 28, 2002 indicate that the peak flow was approximately 15 mgd which confirms the dry weather flow and RDI/I values.

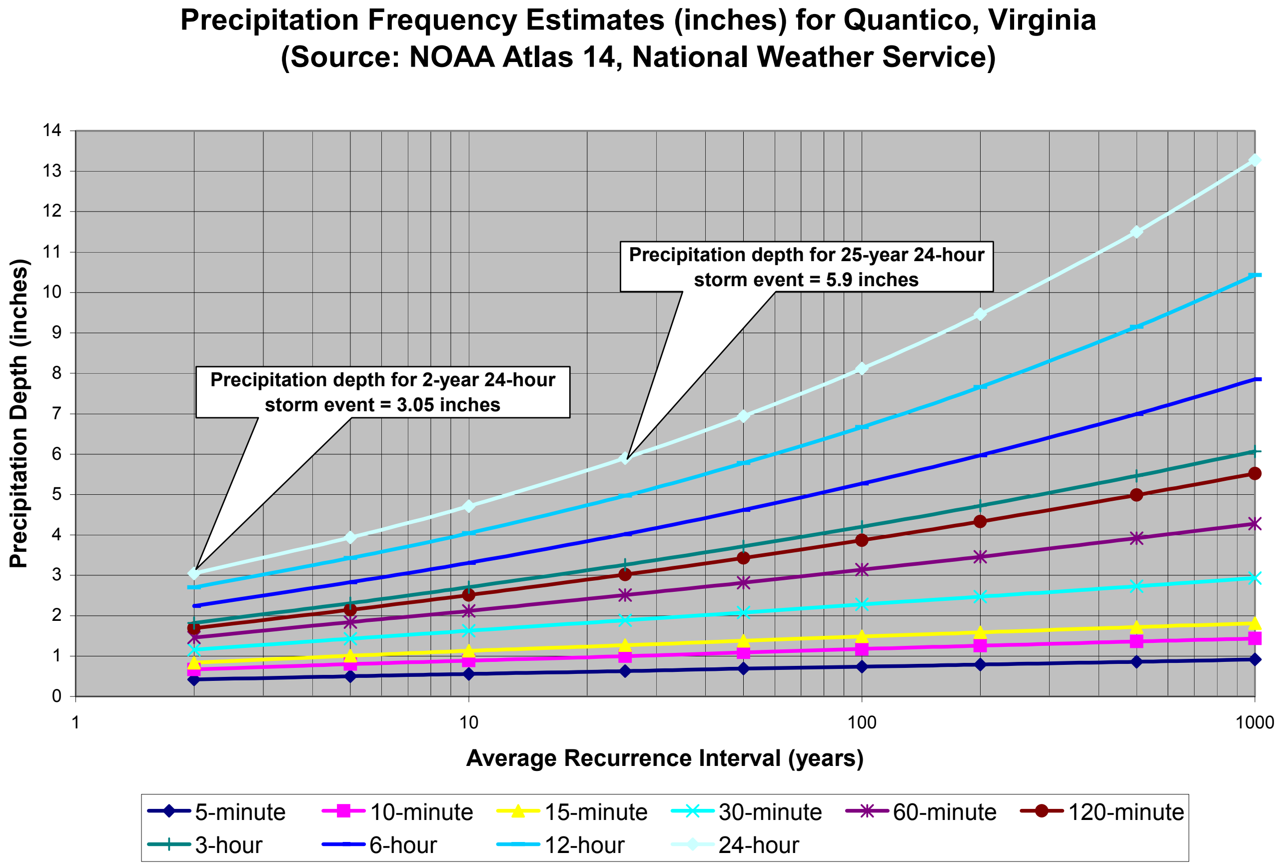
5.0. Findings

Rainfall data and sewer flow monitoring data from the DOU's wastewater conveyance system were needed to calibrate the H2OMAP Sewer model and identify the system's response to storm events of varying characteristics. Groundwater infiltration (GWI) rates were calculated using flow monitoring data from March – April 2003 and August - September 2002. Based on the March through April 2003 results, groundwater infiltration (GWI) rates of 601 to 2,445 gpdidm were exhibited for the Austin/Whitson's Run basin with an average GWI rate of 1,282 gpdidm. For the August through September 2002 data, GWI rates ranged from 222 to 1,478 gpdidm with an average GWI rate of 509 gpdidm. Based on these data and the monthly wastewater treatment plant flow data during the dry weather conditions that occurred between July 2000 and September 2002, a groundwater infiltration rate of approximately 500 gpdidm appears to be a reasonable estimate.

The data from the dry weather and wet weather periods were compared to understand the range of basin peaking factors that existed in the DOU's sewer system and to estimate the amount of rainfall-dependent inflow and infiltration (RDI/I) impacting the different areas of the conveyance system. Based on the August 28, 2002 storm event, SCADA data from eight pumping stations located throughout the sewer system experienced peaking factors ranging from 2.6 to 3.7 (i.e., peak hourly flows were 2.6 to 3.7 times greater than the average daily dry weather flow). For this storm event, the weighted average peaking factor for the system was approximately 2.8.

The data collected indicated that the storms captured during the 2002 and 2003 flow monitoring period had a return interval of 2-years or less. The limited intensity and volume of storms captured will have an impact on the wet weather calibration of the sewer model. Since the storms captured are relatively small, the model may have difficulty simulating larger storm events with longer return intervals. One option is to conduct additional flow monitoring to capture larger storms that can be used to provide additional calibration points.

Rainfall-dependent inflow and infiltration (RDI/I) rates were calculated for each of the flow monitoring locations for the August 28, 2002 storm event. Based on the results of the August 28, 2002 storm event, peak RDI/I rates of 0.31 to 4.47 gpm/manhole were exhibited for the eight pumping stations with 56 percent of the manholes in the sewer system exhibiting a weighted average peak RDI/I rate of 0.74 gpm/manhole (1,065 gpd/manhole).


Precipitation Frequency Estimates (inches) for Quantico, Virginia
(Source: NOAA Atlas 14, National Weather Service)
Precipitation depth for 25-year 24-hour storm event = 5.9 inches
Precipitation depth for 2-year 24-hour storm event = 3.05 inches
Precipitation Depth (inches)
Average Recurrence Interval (years)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
1
10
100
1000
5-minute
10-minute
15-minute
30-minute
60-minute
120-minute
3-hour
6-hour
12-hour
24-hour

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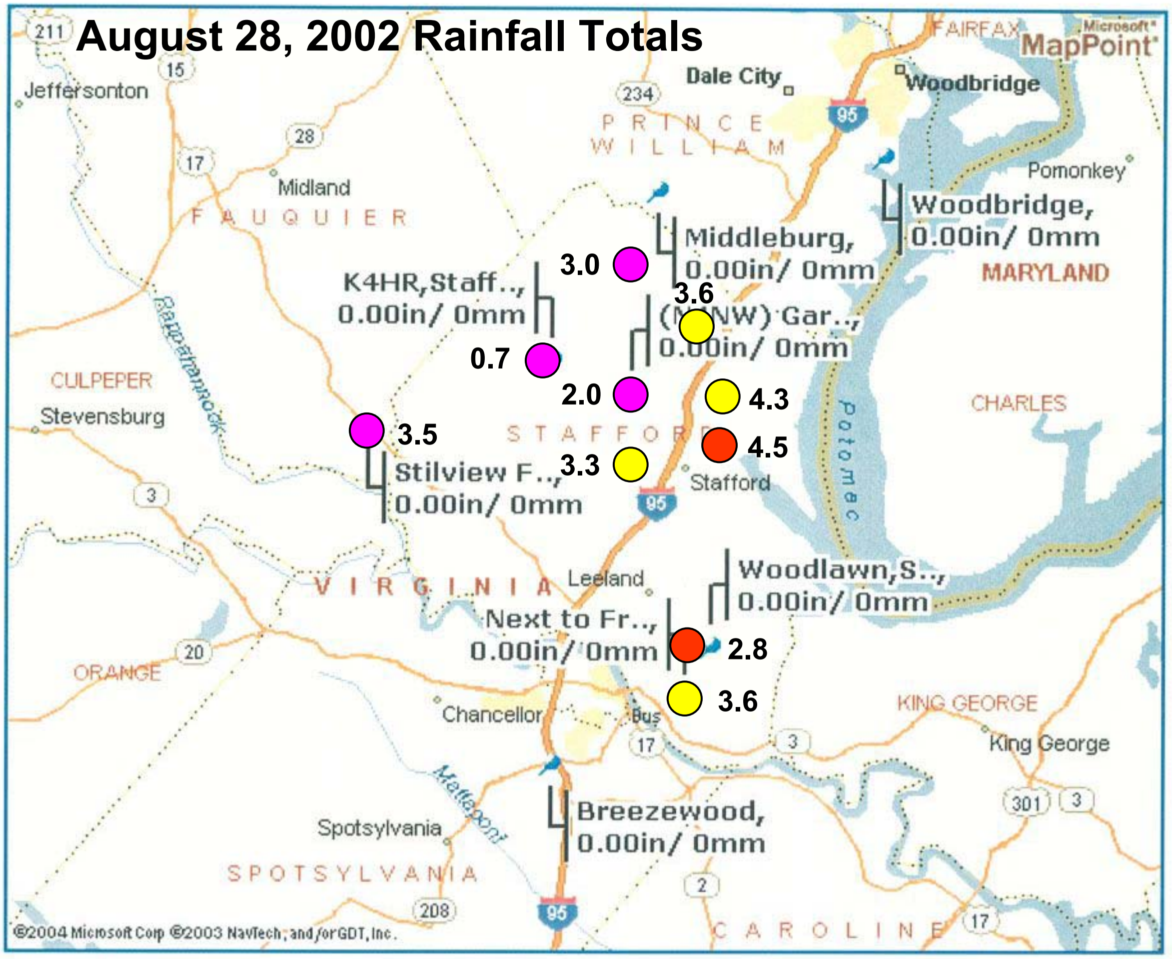

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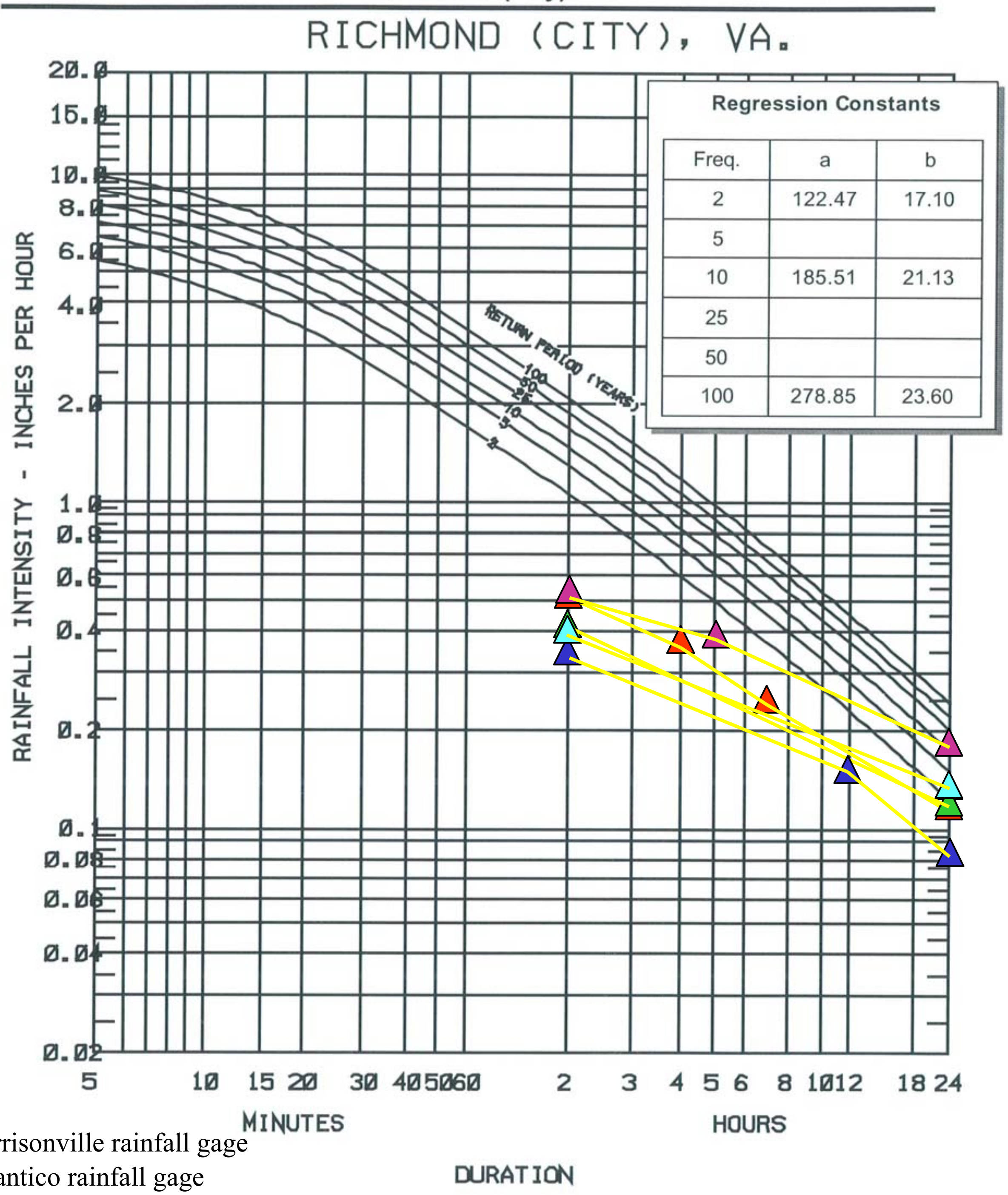
- **Daily Rainfall Totals provided by Stafford County** (yellow)
- **Hourly Rainfall Data from Weather** (magenta)
- **Hourly Rainfall Data from Stafford County** (red)

Rainfall IDF for August 28, 2002

Appendix 6B-12 **Rainfall Intensity Inches Per Hour**
Richmond (City)



Regression Constants

Freq.	a	b
2	122.47	17.10
5		
10	185.51	21.13
25		
50		
100	278.85	23.60

Garrisonville rainfall gage
Quantico rainfall gage
Goldvein rainfall gage

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Daily rainfall total at Abel WTP, Smith Lake WTP, LFR WWTP, and Aquia WWTP
Aquia at Bridge rainfall gage from J. Spencer (Stafford County DOU)

Falls Run rainfall gage from J. Spencer (Stafford County DOU)

TECHNICAL MEMORANDUM 7

Development and Calibration of H2OMAP Sewer Hydraulic Model

Prepared for: Stafford County Department of Utilities
Prepare by: O'Brien & Gere
Date: November 2004

This technical memorandum is one of a series being prepared for the Stafford County Water and Sewer Master Plan project. The purpose of this technical memorandum is to summarize the development and calibration of DOU's sewer system model. This technical memorandum discusses the data gathered as inputs into the model, summarizes the steps necessary to develop and verify the model input data, and outlines the procedures followed to calibrate the model under dry weather and wet weather flow conditions. At the conclusion of the steps described within this technical memorandum, a fully functional, calibrated model was established for DOU's wastewater collection and conveyance system. The calibrated model will be used to identify hydraulic bottlenecks, surcharged pipes, and overflowing manholes simulated within the sewer system under specific flow conditions. The model will be used to evaluate DOU's system in the current year (2003) and in the future (buildout at 2050) to identify the problem areas created by design storm events. Based on the model output, recommendations for improvements to minimize the impacts of problem areas will be documented in Technical Memorandum 8 (*Wastewater Collection, Pumping and Conveyance Facilities*).

This document contains the following sections:

Terminology, Definitions and Glossary

Average Dry Weather Flow (ADWF) – ADWF consists of average daily sewage flows and groundwater infiltration (GWI). ADWF is the average flow that occurs on a daily basis with no evident reaction to rainfall.

C-factor **–** A measure of the interior roughness of a pipe**.**

Diurnal Demand or Flow – Fluctuation of water demands or wastewater flows over a 24-hour period.

Effective Storage – Effective storage for each storage facility is determined by establishing the level in each tank above which all points in the water system can be served at 20 psi or higher (based on peak hour or maximum day plus fire flow).

Equalization Storage – The storage of peaking flows to prevent overflows from the sewer collection and conveyance systems.

Groundwater Infiltration (GWI) – Groundwater that infiltrates pipeline and manhole defects located below the ground surface. Groundwater infiltration is separate and distinguished from inflow resulting from storm events. Infiltration is a steady 24-hour flow that usually varies during the year in relation to the groundwater levels above the sewers. Infiltration rates are normally estimated from wastewater flows measured in the sewers during the early morning hours when water use is at a minimum and the flow is essentially infiltration.

H2OMAP Water – H2OMAP Water is a computer model used for modeling the Department of Utilities' water system under various demand conditions.

H2OMAP Sewer – H2OMAP Sewer is a computer model used for modeling the Department of Utilities' sewer system under various flow conditions.

Inflow – Drainage that enters the collection system through illegal or permitted connections, such as catch basins, downspouts, area drains and manhole covers. Inflow is separate and distinguished from infiltration. The inflow rate can be determined from the flow hydrographs recorded with flow meters by subtracting the normal dry weather flow and the infiltration from the measured flowrate.

Infiltration/Inflow (I/I) – The wastewater component caused by rainfall-dependent infiltration/inflow (RDI/I) and groundwater infiltration (GWI).

Maximum Day Demand **–** The one day in the year when the consumption is the highest.

Maximum Hour Demand **–** The one hour in the year when water consumption is the highest.

Node – A junction of two or more pipes, commonly representing a point where pipe characteristics change.

Peak Dry Weather Flow (PDWF) – PDWF consists of peak sewage flows plus GWI. PDWF is the highest measured hourly flow that occurs on a dry weather day.

Peak Wet Weather Flow (PWWF) – PWWF consists of ADWF plus RDI/I. PWWF is the highest measured hourly flow that occurs during wet weather.

Peak Factor – Peak factor is PWWF/ADWF.

Pressure Reducing Valve (PRV) – A valve that will maintain a specified downstream pressure.

Pressure Zone – A network of water pipes having a common static hydraulic grade line. Pressure zones are separated by closed valves, pressure regulating valves, pumping stations, and reservoirs.

Rainfall-Dependent Infiltration/Inflow (RDI/I) – RDI/I consists of rainfall that enters the collection system through direct connections (roof leaders, manholes, etc.) and causes an almost immediate increase in wastewater flow.

Service Area – The area served by the water distribution or wastewater collection system.

Steady State Simulation – A network model solution for a single point in time.

Tributary Area – The tributary area of a sewage system consists of all areas that contribute flow to the sewer by gravity and/or force main discharges.

ADD	Average Day Demand
ADWF	Average Dry Weather Flow
AWWA	American Water Works Association
CIP	Capital Improvement Program
cfs	Cubic Feet per Second
CMOM	Capacity, Management, Operation and Maintenance
CWA	Clean Water Act
DOU	Stafford County Department of Utilities
D/DBP	Disinfectants/Disinfection Byproducts
EA	Environmental Assessment
EIS	Environmental Impact Statement
EPA	US Environmental Protection Agency
EPS	Extended Period Simulation
ft	Feet
FY	Fiscal Year
gpcpd	Gallons per Capita per Day
gpd	Gallons per Day
gpm	Gallons per Minute
gpdidm	Gallons per Day per Inch Diameter – Mile
GWI	Groundwater Infiltration
HAAs	Haloacetic Acids
HGL	Hydraulic Grade Line
ICR	Information Collection Rule
I/I	Infiltration and Inflow
IESWTR	Interim Enhanced Surface Water Treatment Rule
ISO	Insurance Service Organization
L	Liter
MCL	Maximum Contaminant Level
MDD	Maximum Day Demand

MG	Million Gallons
MGD	Million Gallons Per Day
mg/l	Milligrams per Liter
mgd	Million Gallons per Day
MPN/100 ml	Most Probable Number per 100 Milliliters
NEPA	National Environmental Policy Act
O&M	Operations and Maintenance
PDWF	Peak Dry Weather Flow
PHD	Peak Hour Demand
PRV	Pressure Reducing Valve
psi	Pounds per Square Inch
PSV	Pressure Sustaining Valve
PWWF	Peak Wet Weather Flow
PWS	Public Water Supply
RDI/I	Rainfall-Dependent Infiltration/Inflow
SCADA	Supervisory Control and Data Acquisition
SDWA	Safe Drinking Water Act
SSO	Sanitary Sewer Overflows
SWTR	Surface Water Treatment Rule
TCR	Total Coliform Rule
THMs	Trihalomethanes
UFW	Unaccounted-for Water
ug/L	Micrograms per Liter
USACE	US Army Corps of Engineers
USEPA	US Environmental Protection Agency
USGS	US Geological Survey
VDEQ	Virginia Department of Environmental Quality
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant

Executive Summary

The overall objective for developing and calibrating the hydraulic model of DOU's wastewater collection and conveyance system is to assist in planning and prioritizing future capital improvements program (CIP) projects within DOU's service area. The hydraulic model will be used to simulate flows within the collection system under existing (2003) and future (buildout at 2050) conditions. Under both planning scenarios, design flows will be routed through the model to estimate the impacts to the sewer collection system. Based on the model output of the simulations, recommendations will be developed that meet DOU's collection system loads through buildout.

Development and calibration of the wastewater collection system model proceeded in three phases:

- Phase 1 - Data collection
- Phase 2 - Network development
- Phase 3 - Dry and wet weather calibration

The data collection phase consisted of gathering DOU's best available data on the sewer collection and conveyance system to be modeled. The model network that was developed consisted of gravity pipelines, as well as pumping stations and force mains. Specific data on the components of the network were gathered and incorporated into the H2OMAP Sewer model by DOU. Sewer connectivity was based on information from DOU's Geographic Information System (GIS) and system mapping.

Wastewater inflows used in the model were based on sanitary base flows, groundwater infiltration and rainfall-dependent infiltration/inflow. During model development, DOU's existing and future land uses were used to generate sanitary base flows at the sewer system manholes. Using the GIS, the sanitary base flows at each manhole were computed by taking the sewer flows for various land uses in the sewer service area and assigning sewer flows to the nearest sewer manhole serving the tributary area. Average dry weather flows included the sanitary base flow generated for each manhole from the land use and the groundwater infiltration (GWI). Wet weather flows were composed of dry weather flows plus rainfall dependent I/I (RDI/I). The 2002 and 2003 flow monitoring data were used to estimate the amount of dry and wet weather flow introduced to the sanitary sewer system.

Calibration of the sewer model was necessary under both dry and wet weather conditions. Dry and wet weather calibration at several locations in DOU's sewer system demonstrated significant correlation between the modeled flow and the actual measured flow collected in 2002 and 2003.

Upon completion of the tasks described in this technical memorandum, the wet weather calibration of the hydraulic model of DOU's wastewater collection system was completed for the storm that was captured on August 28, 2002 (e.g., approximately a 2-year storm event). For the model to be calibrated for larger storms with longer return intervals, additional flow monitoring data will need to be collected and run through the hydraulic model. It is recommended that DOU consider collecting additional rainfall and flow monitoring data in the future to allow collection of data from larger storms which can be used to recalibrate the model and better predict the response and impact from a wider variety of rainfall events.

1.0. Data Collection

The hydraulic model of the DOU's wastewater collection and conveyance system was developed by DOU using the best available data. For the existing sanitary sewer system, DOU developed information on pumping stations, pipes, manholes, and control structures which served as the physical foundation of the sewer model. In addition to the physical data, current and future sewer loads were needed. Data from the sewage pumping stations (DOU SCADA system data), wastewater treatment plant influent data, and data from the 2002 and 2003 flow monitoring programs were gathered and analyzed to assist in model calibration.

1.1. Pipe Network Data

The sewer system model was obtained from DOU and was based on an inventory of sewer piping and facilities identified in DOU's Geographic Information System. Pipes in the model are represented by line segments and are defined by an upstream manhole, a downstream segment of pipe and a downstream manhole. Most models consider manholes and wetwells as "nodes", and pipes, force mains, pumping stations, and control structures as "links".

The hydraulic model of DOU's wastewater collection and conveyance system includes gravity flow pipes and manholes. The key data for the pipes and manholes in the model include:

Pipes (links)

Pipe name
Upstream manhole
Downstream manhole
Length
Cross section type
Pipe diameter
Upstream invert elevation
Downstream invert elevation

Manholes (nodes)

Manhole name
Ground surface elevation
Manhole invert elevation
X coordinate
Y coordinate

DOU provided these data in the configured sewer model.

1.2. Pumping Station Data

In addition to the sewer piping and corresponding manholes in the sewer network, pumping stations and their corresponding force mains in DOU's sewer system were also included in the model. DOU currently operates 83 pumping stations located throughout the sanitary sewer system.

The information in the model for pumping stations includes station location, number and size of pumping units, total capacity (all pumps operational) of the station, firm capacity of the station, size of the wetwell, and inlet and discharge elevations. The firm capacity of the station is defined as the capacity of the pumping station when the largest capacity pump is out-of-service.

1.3. Wastewater Inflows

Average sanitary base flows for current (2001) conditions for the DOU service area were based on an 80% reduction of the water meter billing data (2001) for the customers in the sewer service area. Water demands were used to generate sewer loads by reducing the water demands and allocating the demands to nearest nodes (manholes).

1.4. Flow Monitoring Data

In March through April 2003, DOU installed ten temporary flow meters in the wastewater conveyance system for 35 days to gather flow data within Austin/Whitson's Run Basin as part of an inflow/infiltration study. Rain gage data from Aquia WWTP and Quantico Marine Corps Base were used in the I/I study. The data from each of the ten flow meters and SCADA data from four pumping stations in the Austin/Whitson's Run basin were reviewed, analyzed and summarized under dry and wet weather conditions during this period. In addition to the March through April 2003 flow monitoring data for

Austin/Whitson's Run, flow data from the DOU's SCADA system were obtained for eight pumping stations for a 35-day period from August 22 through September 25, 2002.

The data from the flow meters and pumping stations were reviewed, analyzed and summarized under dry and wet weather flow conditions, as presented in Technical Memorandum 6 *(Rainfall/Flow Monitoring Program)*. The flow monitoring data were used for calibration of the sewer model under dry and wet weather conditions.

2.0. Model Development

In general, DOU established the physical input data for the uncalibrated sewer model prior to initiation of the Master Plan. Sewer loads to be routed through the sewer system were established during the Master Plan.

2.1. Pipe Network Data and Connectivity

Pipes are conduits that transport flow through the sewer system either by gravity (i.e., gravity mains) or by the energy supplied from pumps (i.e., force mains). DOU staff performed quality control checks on the pipe network data and connectivity during model construction. In addition, the H2OMAP Sewer software performs a number of quality control checks on the system during model applications, including checks on the connectivity of the sewer system.

2.2. Sewer Loads

Sewer loads at manholes consist of sanitary base flow and flows resulting from rainfall events for the area tributary to the manhole. Groundwater infiltration is applied to the segments of pipe in the sanitary sewer system between the manholes. These sewer loads and infiltration represent estimates of the amount of wastewater flow that must be handled by the collection and conveyance system. The location in the hydraulic model for introducing the sewer load depends on the type of sewer load:

- <u>Sanitary base flow</u> - applied at manholes in the model of the sewer system.
- <u>Groundwater infiltration (GWI)</u> – applied to each segment of pipe in the sewer model.
- <u>Rainfall-dependent I/I (RDI/I)</u> - applied at manholes in the model of the sewer system.

DOU's GIS sewer layer and hydraulic model network of the sewer system were used as the basis for delineating the existing sewer service area. The service area boundary for future conditions (buildout) was based on the existing sewer service area, projected land use, sewershed boundaries (i.e., drainage basins, roadway and water features, etc.) and discussions with DOU and Planning Department staff regarding future development and policies. Once the sewer service areas were delineated, sewer loads were input by using a feature of the modeling software that compiles sewer loads and assigning the loads to the nearest manholes in the H2OMAP Sewer model using a polygon area surrounding the manhole. This approach results in an accurate allocation of existing sewer flows based on existing water meter billing data.

3.0. Sewer Loads and Calibration for Dry Weather Conditions

Dry weather flow conditions occur when rainfall is not influencing flows in the sewer system. Dry weather flow in a sewer system is composed of two components:

- Sanitary base flow generated by homes, businesses, etc.
- Infiltration due to normal groundwater levels (dry weather infiltration).

To reasonably simulate the hydraulics of the sewer system, the flow is separated based on the source of the flow. The dry weather components of the wastewater flow (sanitary base flow and groundwater infiltration) were generated first. The sanitary base flows at each manhole were derived from the land use tributary to the manhole. The groundwater infiltration (GWI) was estimated using data from the 2002 and 2003 flow monitoring program.

During dry weather conditions, flow in the sewer system is the sum of the sanitary base flow and groundwater infiltration (GWI). The dry weather flow at a point in the sewer system can be calculated as follows:

Average Dry Weather Flow (ADWF) = Sanitary Base Flow + Groundwater Infiltration (GWI)

Where:

Average Dry Weather Flow (ADWF) is the average flow that occurs in the sanitary sewer on a daily basis with no evident reaction to rainfall.

Sanitary base flow equals the average daily water demand based on water duties presented in Technical Memorandum 2 (Water Demands) multiplied by a percentage reduction which is an estimate of the customer water demand that is returned to the sewer.

Groundwater infiltration (GWI) is an allowance that is added to the sanitary base flow (derived from sewage flow factors) to obtain the dry weather flow. GWI represents flow that is separate and distinguished from inflow resulting from storm events during wet weather conditions.

The components presented in the formula are outlined below.

3.1. Sanitary Base Flow

The sanitary base flow at each manhole represents the sewer loads that are assigned to the manhole. While it is difficult to estimate the portion of water use that is reaching the sanitary sewer system (return flow), an estimate of domestic, industrial and commercial wastewater flow rates can be subtracted from the total flow measured at the wastewater treatment plants to obtain an estimate of the infiltration entering the system. For most sewer systems, the portion of water reaching the sanitary sewer system typically ranges from 80% to 90%.

3.1.1. Sewer Loads from Customer Use

Wastewater treatment plant flow data for July 2000 through December 2002 were used to check the reasonableness of the sanitary base flow estimate of 64 gallons per day per person (80% of 80 gpd/person of water demand). Based on an average monthly wastewater flow of approximately 6 mgd for the period from July 2000 through December 2002 and approximately 63,000 customers, the average wastewater generation rate is roughly 95 gallons per person per day. Using 3,477 inch diameter-mile for the existing overall sewer system (not including service connections) and 500 gpdidm for the GWI rate yields approximately 1.74 mgd of GWI which is roughly 28 gpd/person assuming 63,000 customers. Subtracting the GWI rate of 28 gpd/person from the total per capita flow of 95 gpd yields a sanitary base flow of approximately 67 gpd/person. This flow closely approximates the estimate of 64 gallons per day per person. In addition, the American Water Works Association identifies that indoor water use that would likely be returned to the sewer system typically ranges from 60 to 65 gpd/person (American Water

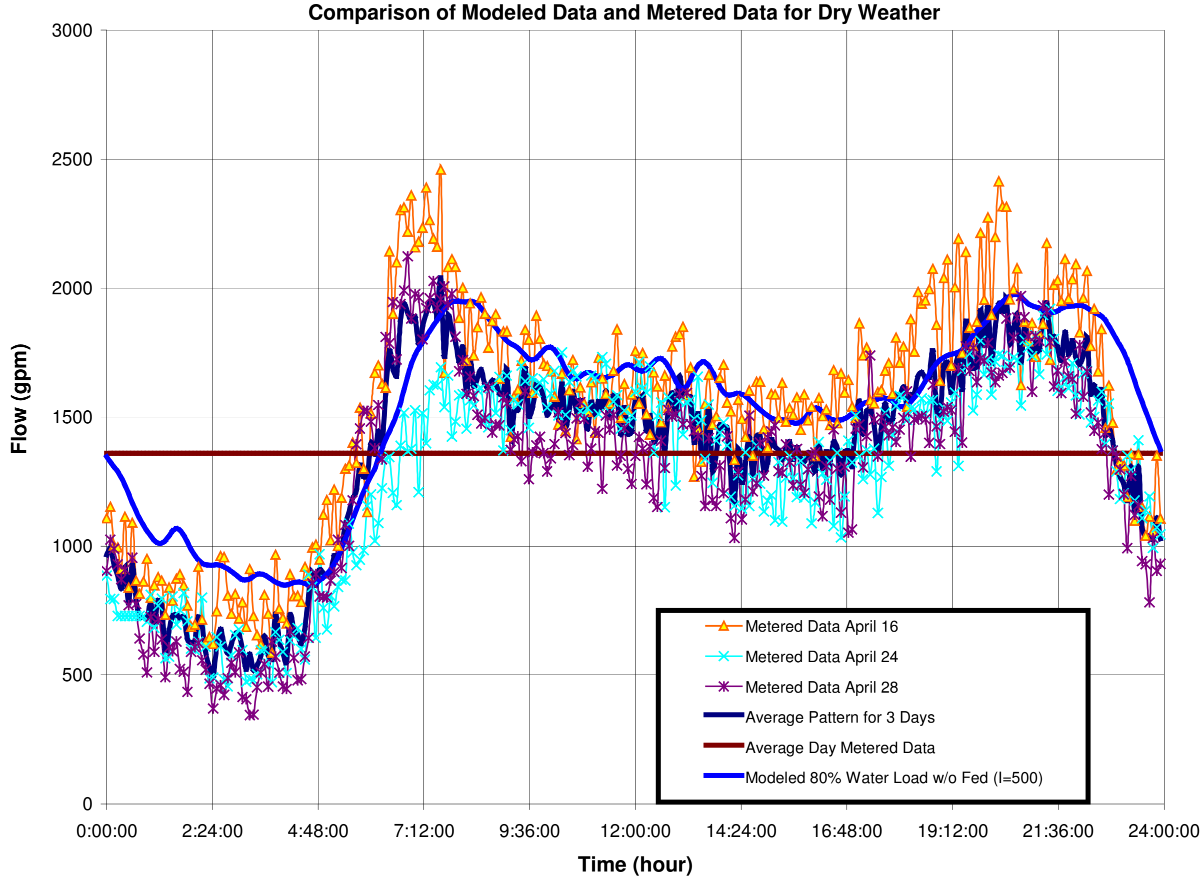
Works Association, Residential End Uses of Water, 1999). Consequently, using a sanitary base flow of 64 gpd/person appears to be reasonable.

3.1.2. Sewer Load Patterns for Dry Weather Conditions

Sanitary base flows in sewer systems vary throughout the day with peaks in the morning and evening and low flows in the early morning hours. Patterns are used to represent the daily temporal variations within the sewer system. The patterns consist of a collection of multipliers (multiplication factors) that are applied to the sanitary base load to allow it to vary over time during an extended period simulation (EPS). At any point during a day, the load at a manhole under dry weather conditions is the sanitary base load multiplied by the current pattern multiplier. Different patterns can be applied to individual manholes or groups of manholes to accurately represent loading categories (e.g., residential, commercial, etc.). For the calibration analysis, data from the 2003 flow monitoring program in Austin/Whitson's Run were used to develop a dry weather flow pattern that was applied to manholes in the Austin/Whitson's Run system (Figure 1).

Figure 1: Gravity Main P40-0105A & Manhole 40-0105

Comparison of Modeled Data and Metered Data for Dry Weather



The dry weather flow pattern consists of the combined loading categories (i.e., separate patterns for various land use types such as residential and commercial were not generated in this study). Flow monitoring data collected from the interceptor on Austin Run in the vicinity of Basin 8 were used to generate the dry weather pattern for calibration. The dry weather pattern was developed using three days of dry weather (April 16, 24, and 28, 2003) that were preceded by dry weather periods of at least a few days. The dry weather flow pattern was based on average factors every 30 minutes (pattern timestep in model) over a 24-hour period (duration in model). The dry weather pattern was considered to be uniform throughout the sewer system upstream of the flow monitoring point. Consequently, sanitary base loads upstream of the calibration points were multiplied by the dry weather pattern for the dry weather calibration. Dry weather flow patterns generated in Figure 1 were used for the entire system and were used for calibrating the model.

3.2. Estimation of Groundwater Infiltration

As discussed in Technical Memorandum 6 (*Rainfall/Flow Monitoring Program*), data from August through September 2002 indicated that the GWI rates ranged from 222 to 1,478 gpdidm with an average GWI rate of 509 gpdidm. Based on these data and the monthly wastewater treatment plant flow data during the dry weather conditions that occurred between July 2000 and September 2002, a groundwater infiltration rate of approximately 500 gpdidm appears to be a reasonable estimate and was used in this Master Plan.

Infiltration rates were input as constant flow sources within the hydraulics layer of the model. Because it is unknown where infiltration enters the sanitary sewer system without extensive testing (i.e., night flow isolation and measurement), the GWI rate of 500 gpdidm was developed and used for the entire model. It is important to note that the infiltration rate is applied to the pipes in the model to produce a steady, unpeaked flow.

3.3. Average Dry Weather Flow

The sanitary base flow was combined with the groundwater infiltration to obtain the average dry weather flow. The average dry weather flow at a point in the sewer system can be calculated as follows:

Average Dry Weather Flow (ADWF) = Sanitary Base Flow + Groundwater Infiltration (GWI)

Where:

Average Dry Weather Flow (ADWF) is the average flow that occurs in the sanitary sewer on a daily basis with no evident reaction to rainfall.

Sanitary base flow equals the average daily water demand based on water duties presented in Technical Memorandum 2 – Water Demands multiplied by 80% which is an estimate of the customer water demand that is returned to the sewer system.

Groundwater infiltration (GWI) is an allowance that is added to the sanitary base flow (derived from sewage flow factors) to obtain the dry weather flow. GWI represents flow that is separate and distinguished from inflow resulting from storm events during wet weather conditions. The GWI rate used in the Master Plan is 500 gpdidm.

Using the dry weather flow data from 2002 and 2003, flow patterns for the average dry weather day were generated for flow monitoring points throughout the system. Calibration under dry weather flow conditions was performed to verify the sanitary base flows generated. Calibration under wet weather conditions could not be completed until after dry weather calibration because the dry weather flows were the foundation for the sewer loads at the manholes in the model. Calibration was performed at several locations using the data from the 2002 and 2003 flow monitoring program. The primary goal of the calibration was to match the volume of flow generated in the model with the volume measured during monitoring. The secondary goal was to match the average dry weather flow pattern between the two data sets.

During model calibration, it is necessary to achieve a reasonable match between observed and modeled peak flow, time of concentration, and total volume. The accuracy of calibration is often best visualized through use of flow hydrographs. Calibration data were input to the sewer model and the results for Austin/Whitson's Run are shown on Figure 1 for the flow monitoring point near Basin 8. As shown on Figure 1, using 80% of the water demand for the upstream sewer basin along with a GWI rate of 500 gpdidm produces model flows that closely match the peaks and the pattern of the average flow measured for the three dry weather days in August 2002. Therefore, these dry weather components appear to be accurate estimates and the model is considered calibrated for dry weather conditions.

4.0. Sewer Loads and Calibration for Wet Weather Conditions

The formula for calculating the sewer loads for wet weather conditions is as follows:

Peak Wet Weather Flow (PWWF) = Average Dry Weather Flow (ADWF) + RDI/I

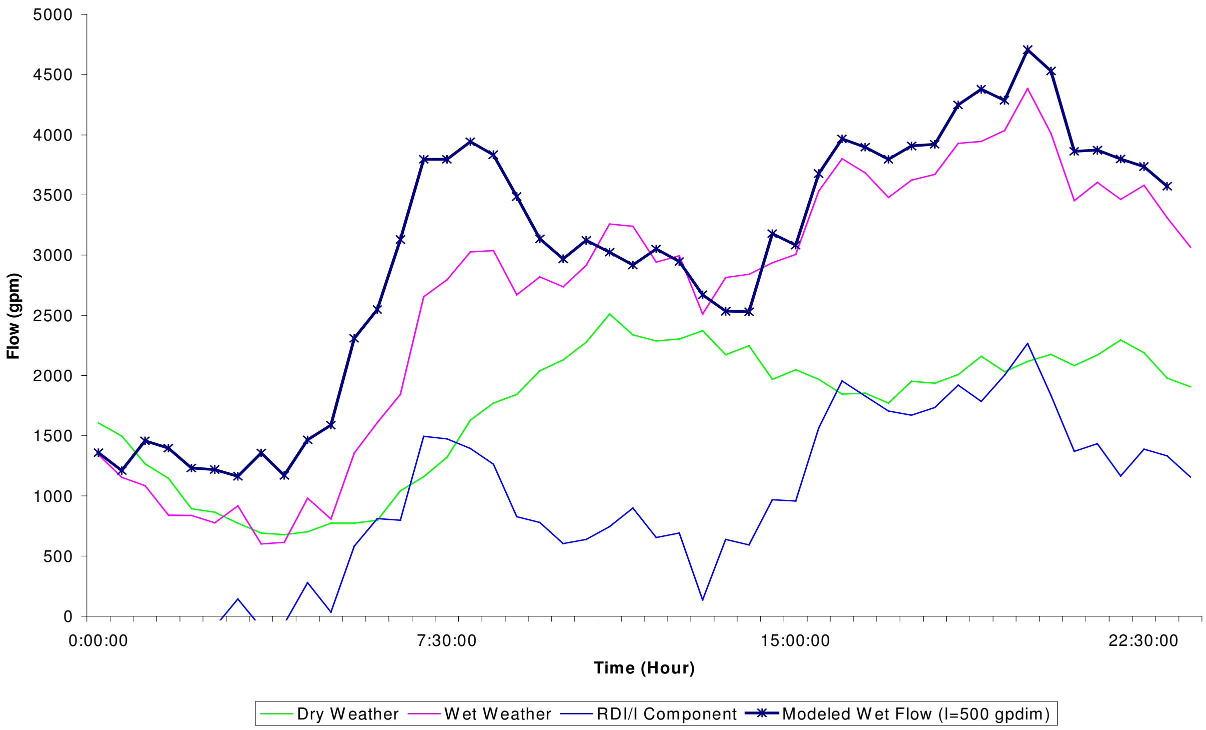
The rainfall-dependent infiltration/inflow (RDI/I) component was estimated using dry weather and storm event data from the 2002 and 2003 flow monitoring program (Technical Memorandum 6 – Rainfall/Flow Monitoring Program).

Wet weather flow calibration began after dry weather calibration was completed. The purpose of wet weather calibration is to prepare a model to handle the inflows created by rainfall events (i.e., rainfall-dependent I/I). The ultimate goal of the wet weather flow calibration was for the modeled data to match the storm peaks from the 2002 flow monitoring data. The storm event used for wet weather calibration occurred on August 28, 2002.

4.1. Comparison of Wet Weather Flow Projections with Flow Monitoring Data

Rainfall-dependent I/I can be defined as rainwater that enters a sanitary sewer collection system and causes an almost immediate increase in wastewater flows. RDI/I is generally more difficult to define than infiltration as it is specific to each individual storm event. Under wet weather flow conditions, RDI/I was modeled as a response to rainfall and in the form of a hydrograph. The RDI/I (inflow) hydrograph for the August 28, 2002 storm event was determined by subtracting the hourly metered flows at a location from the corresponding hourly values comprising the diurnal curve for dry weather conditions at that location as shown in Figure 2.

Figure 2: Austin Run Sewer Service Area
Comparison of Metered to Modeled Data (Wet Weather on Aug 28, 2002)



For the wet weather model calibration, RDI/I was assumed to be uniform upstream of the flow monitoring location. An RDI/I hydrograph was generated at each manhole upstream of the flow monitoring location by dividing the measured flow at the monitoring point for each hour in the 24-hour period by the number of upstream manholes (gpm/manhole). Wet weather flows were generated by combining the RDI/I hydrograph data and the dry weather flow curve data for each hour during the 24–hour period.

4.2. Results of Wet Weather Flow Calibration

The primary focus of the wet weather flow calibration was to match the peak flows at the selected flow monitoring locations during the storm event that was captured in August 2002. As shown on Figure 2, the model flows closely match the peaks and the pattern of the average flow measured for the August 28, 2002 storm event. Based on the correlation demonstrated by Figures 1 and 2, it was determined that the hydraulic model of DOU's wastewater collection system was calibrated up to the size of the storm captured during the 2002 flow monitoring program. The August 28, 2002 storm event was approximately a 2-year event. When storm events with more than a 2-year return interval were simulated within the model, the same wet weather parameters were used to estimate flow and, ultimately, impacts to the system. The model was not calibrated for the larger storms because no monitoring data were available upon which to calibrate. The model extrapolated the characteristics and responses demonstrated in the smaller storms to the larger storms, but there is no way to verify how the system would actually respond.

TECHNICAL MEMORANDUM 8

Wastewater Collection, Pumping and Conveyance Facilities

Prepared for:	Stafford County Department of Utilities
Prepare by:	O'Brien & Gere
Date:	November 2004 (Revised May 2005)

This technical memorandum is one of a series being prepared for the Stafford County Water and Sewer Master Plan project. The purpose of this technical memorandum is to document specific sewer system upgrade and expansion options that can be implemented to meet DOU's wastewater collection and conveyance demands through the buildout planning horizon. The specific recommendations are based on results from the following:

- Calibrated hydraulic model developed for DOU's wastewater collection and conveyance system.
- Infiltration and inflow (I/I) analysis conducted as part of the 2002 and 2003 monitoring program.
- Sewer flows based on proposed developments presented in Appendix A.
- Buildout flows based upon the County's Land Use Plan

This document contains the following sections:

Terminology, Definitions and Glossary

Average Dry Weather Flow (ADWF) – ADWF consists of average daily sewage flows and groundwater infiltration (GWI). ADWF is the average flow that occurs on a daily basis with no evident reaction to rainfall.

C-factor – A measure of the interior roughness of a pipe.

Diurnal Demand or Flow – Fluctuation of water demands or wastewater flows over a 24-hour period.

Effective Storage – Effective storage for each storage facility is determined by establishing the level in each tank above which all points in the water system can be served at 20 psi or higher (based on peak hour or maximum day plus fire flow).

Equalization Storage – The storage of peaking flows to prevent overflows from the sewer collection and conveyance systems.

Groundwater Infiltration (GWI) – Groundwater that infiltrates pipeline and manhole defects located below the ground surface. Groundwater infiltration is separate and distinguished from inflow resulting from storm events. Infiltration is a steady 24-hour flow that usually varies during the year in relation to the groundwater levels above the sewers. Infiltration rates are normally estimated from wastewater flows measured in the sewers during the early morning hours when water use is at a minimum and the flow is essentially infiltration.

H2OMAP Water – H2OMAP Water is a computer model used for modeling the Department of Utilities' water system under various demand conditions.

H2OMAP Sewer – H2OMAP Sewer is a computer model used for modeling the Department of Utilities' sewer system under various flow conditions.

Inflow – Drainage that enters the collection system through illegal or permitted connections, such as catch basins, downspouts, area drains and manhole covers. Inflow is separate and distinguished from infiltration. The inflow rate can be determined from the flow hydrographs recorded with flow meters by subtracting the normal dry weather flow and the infiltration from the measured flowrate.

Infiltration/Inflow (I/I) – The wastewater component caused by rainfall-dependent infiltration/inflow (RDI/I) and groundwater infiltration (GWI).

Maximum Day Demand – The one day in the year when the consumption is the highest.

Maximum Hour Demand – The one hour in the year when water consumption is the highest.

Node – A junction of two or more pipes, commonly representing a point where pipe characteristics change.

Peak Dry Weather Flow (PDWF) – PDWF consists of peak sewage flows plus GWI. PDWF is the highest measured hourly flow that occurs on a dry weather day.

Peak Wet Weather Flow (PWWF) – PWWF consists of ADWF plus RDI/I. PWWF is the highest measured hourly flow that occurs during wet weather.

Peak Factor – Peak factor is PWWF/ADWF.

Pressure Reducing Valve (PRV) – A valve that will maintain a specified downstream pressure.

Pressure Zone – A network of water pipes having a common static hydraulic grade line. Pressure zones are separated by closed valves, pressure regulating valves, pumping stations, and reservoirs.

Rainfall-Dependent Infiltration/Inflow (RDI/I) – RDI/I consists of rainfall that enters the collection system through direct connections (roof leaders, manholes, etc.) and causes an almost immediate increase in wastewater flow.

Service Area – The area served by the water distribution or wastewater collection system.

Steady State Simulation – A network model solution for a single point in time.

Tributary Area – The tributary area of a sewage system consists of all areas that contribute flow to the sewer by gravity and/or force main discharges.

ADD	Average Day Demand
ADWF	Average Dry Weather Flow
AWWA	American Water Works Association
CIP	Capital Improvement Program
cfs	Cubic Feet per Second
CMOM	Capacity, Management, Operation and Maintenance
CWA	Clean Water Act
DOU	Stafford County Department of Utilities
D/DBP	Disinfectants/Disinfection Byproducts
EA	Environmental Assessment
EIS	Environmental Impact Statement
EPA	US Environmental Protection Agency
EPS	Extended Period Simulation
ft	Feet
FY	Fiscal Year
gpcpd	Gallons per Capita per Day
gpd	Gallons per Day
gpm	Gallons per Minute
gpdidm	Gallons per Day per Inch Diameter – Mile
GWI	Groundwater Infiltration
HAAs	Haloacetic Acids
HGL	Hydraulic Grade Line
ICR	Information Collection Rule
I/I	Infiltration and Inflow
IESWTR	Interim Enhanced Surface Water Treatment Rule
ISO	Insurance Service Organization
L	Liter
MCL	Maximum Contaminant Level
MDD	Maximum Day Demand
MG	Million Gallons
MGD	Million Gallons Per Day

mg/l	Milligrams per Liter
mgd	Million Gallons per Day
MPN/100 ml	Most Probable Number per 100 Milliliters
NEPA	National Environmental Policy Act
O&M	Operations and Maintenance
PDWF	Peak Dry Weather Flow
PHD	Peak Hour Demand
PRV	Pressure Reducing Valve
psi	Pounds per Square Inch
PSV	Pressure Sustaining Valve
PWWF	Peak Wet Weather Flow
PWS	Public Water Supply
RDI/I	Rainfall-Dependent Infiltration/Inflow
SCADA	Supervisory Control and Data Acquisition
SDWA	Safe Drinking Water Act
SSO	Sanitary Sewer Overflows
SWTR	Surface Water Treatment Rule
TCR	Total Coliform Rule
THMs	Trihalomethanes
UFW	Unaccounted-for Water
ug/L	Micrograms per Liter
USACE	US Army Corps of Engineers
USEPA	US Environmental Protection Agency
USGS	US Geological Survey
VDEQ	Virginia Department of Environmental Quality
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant

Executive Summary

For the DOU's Master Plan, the two key sources of information used to assess the performance of the collection system and hydraulic restrictions were:

- 2002 and 2003 flow monitoring data and infiltration and inflow (I/I) analysis.
- Hydraulic sewer model of pipes and pumping stations.

These information sources and the hydraulic modeling tool were used to analyze the system and assess the performance characteristics.

Overview of DOU's Sewer System

DOU's wastewater collection and conveyance system is served by two wastewater treatment plants:

- Aquia WWTP in the northern portion of the service area along Austin Run and adjacent to Jefferson Davis Highway.
- Little Falls Run WWTP in the southeastern portion of the County along Kings Highway and near the confluence of Little Falls Run and the Rappahannock River.

The DOU wastewater collection system consists of approximately 341 miles of gravity pipe, 47 miles of force mains, 8,673 manholes, and 83 pumping stations. Pipe sizes in the collection system range from 6 to 36 inches in diameter.

Regulatory Considerations and Level of Service Requirements

Sanitary sewer overflows (SSOs) are discharges of untreated sewage from a municipal sanitary sewer system prior to the headworks of the WWTP. SSO discharges are prohibited under the Clean Water Act unless authorized by an NDPES permit. The United States Environmental Protection Agency (USEPA) and the Virginia Department of Environmental Quality (VDEQ) believe that inadequate management, operation and maintenance of sewage collection and conveyance systems pose a significant threat to receiving water quality and public health through the discharge of untreated waste into the environment.

The USEPA and VDEQ acknowledge that SSOs cannot be completely eliminated, and that sanitary sewer systems that are designed not to overflow when a given design storm occurs, may nonetheless experience wet weather induced overflows as the result of conditions other than the design storm. Therefore, as part of the NPDES permitting process, it is anticipated that USEPA and VDEQ will require certification that DOU's sanitary sewer system will not experience SSO events as a result of storm events equal to, or less than, a design storm of specified intensity and duration. The USEPA and VDEQ have not yet defined the design storm criteria; however, DOU's collection and conveyance system had been analyzed for impacts associated with a peak flow equal to 3.5 times the average dry weather flow which equates to a 25-year peak inflow event.

Sewer Loads and Design Storm Events

Design flow for a sewer is defined as the maximum flow rate that occurs under selected weather and growth conditions. Since a significant portion of the peak flows result from rainfall, the design storm flow that the sewer must convey is related to the probability of occurrence of a design storm event. Design flow for a selected rainfall event is the sum of the peak sanitary base flow, infiltration and inflow.

The design storm or storm recurrence interval is also the basis for prescribing a level of protection to the pipe capacity to carry the design flow. The selection of design storm becomes an integral component of the Capacity Assurance Plan (CAP) since it determines the threshold flows at which the sewer will be expected to surcharge and potentially overflow. For this Master Plan, DOU's collection and conveyance system had been analyzed for impacts associated with a peak flow equal to 3.5 times the average dry weather flow which equates to a 25-year peak inflow event. The sewer flows associated with current and buildout conditions are presented in Table 1.

Table 1: Current and projected sewer loads

Year	Average Day Sewer Load (mgd)	Peak Flow (mgd)
Current (2001)	6.0	21
Future (buildout at 2050)	19.8	69.4

Hydraulic Peaking Factors

Over time, DOU will continue to implement I/I reduction projects that will likely reduce the amount of rain and groundwater that enters the collection system, thereby reducing the hydraulic peaking factors observed at the Aquia and Little Falls WWTPs. Conversely, during the same time period, DOU will implement projects to add conveyance capacity (either larger pipes or added pumping capacity) to address potential overflow conditions in the collection system; thereby increasing the hydraulic peaking factors at the Aquia and Little Falls Run WWTPs. Depending on the nature and degree of capacity improvements in the collection system, more or larger process units may be required at the WWTPs. As DOU continues to identify alternatives to improve the operation and performance of the collection system, it will be important to correlate these improvement project with potential impacts on hydraulic peaking factors observed at the WWTPs.

Evaluation Criteria

The design criteria curves presented in this technical memorandum were used to evaluate the capabilities of the sanitary sewer system under steady state conditions. The analysis and design criteria curves presented in this memorandum are proposed as the basis for identifying the sanitary sewer system deficiencies and future improvements. Modeling runs using alternative analysis and design criteria curves, as well as peaking factors, were performed to assess the impact the criteria have on the need for and timing of future improvements. Decisions regarding the need to implement the identified future improvements will require consideration on a case-by-case basis taking into consideration a number of factors, such as remaining development potential.

Hydraulic Modeling Tool

A fully functional, calibrated model was developed for the DOU wastewater collection and conveyance system. The hydraulic model can be used to better understand and assess the capacity of the DOU collection and conveyance system by simulating and identifying hydraulic restrictions – surcharging pipes and overflowing manholes – within the system under specified flow conditions. The model was calibrated for current (2003) flows under dry and wet weather conditions. The wet weather calibration was based on data from the 2002 and 2003 flow monitoring program. It is important to note that the model is only calibrated for the storm events that occurred during the flow monitoring program. To improve the predictive capabilities of the hydraulic model over larger ranges of wet weather inflow conditions, DOU should consider continuing to conduct rainfall and flow monitoring activities to capture additional storm events with varying characteristics (intensity, duration and volume).

Key Issues and Challenges

This Master Plan addresses the hydraulic performance of sewer system and the timing for improvements. However, it is recognized that rehabilitation and replacement of sewer infrastructure could alter the timing for the improvements identified in this Master Plan. Essentially all infrastructure systems, including sewer systems, continually deteriorate starting from the day the system is placed into service.

Over time, many biological, chemical and physical forces act on the sewer pipes to reduce their integrity, including but not limited to, the following:

- Construction activity above, or close to, sewer lines can cause the ground to shift, which can open gaps at joints, cause pipes to become misaligned, or cause breaks and collapses.
- Groundwater can undermine pipes causing them to sag and open joints.
- Hydrogen sulfide produced in the sewer can corrode the crown of concrete pipe.
- Root intrusion at joints and cracks can wedge open pipes.
- Poorly made lateral connections can weaken a pipe or leave a gap around the connection.

Field investigations have shown that all pipe materials are subject to some form of biological, chemical or physical deterioration, and all sewers will eventually fail and require rehabilitation and/or replacement. The challenge to wastewater utilities is to be able to forecast and plan future rehabilitation and replacement projects with sufficient accuracy and time in order to minimize the number and frequency of unscheduled maintenance activities and enhance the operation and performance of the system.

Recommended Improvements

In keeping with the strategy DOU has adopted for the wastewater collection system, capital improvement program (CIP) recommendations have been developed to address hydraulic capacity deficiencies. The capital program outlined in this Master Plan has a total cost of approximately $76 million for the improvements needed though buildout (2050). A map showing the proposed improvements and the summary of the cost and timing of improvements are included in the pockets at the end of this Master Plan.

1.0. Overview of Existing System

DOU's wastewater collection and conveyance system is served by two wastewater treatment plants:

- Aquia WWTP serves the northern portion of the service area and is located along Austin Run and near the Jefferson Davis Highway and Coal Landing Road intersection.
- Little Falls Run WWTP serves the southeastern portion of the County and is located on Kings Highway just east of the confluence of Little Falls Run and the Rappahannock River.

The DOU wastewater collection system consists of approximately 341 miles of gravity sewer pipe, 47 miles of force mains, 8,673 manholes, and 83 pumping stations. Pipe sizes in the collection system range from 6 to 36 inches in diameter. The most common pipe materials in the collection and conveyance system are reinforced concrete pipe (RCP), cast iron pipe (CIP), ductile iron pipe (DIP), polyvinyl chloride (PVC), and asbestos cement pipe (ACP). Prior to 1978, ACP was primarily used. In more recent construction, PVC pipe has been used extensively. The first conventional wastewater collection facilities in Stafford County were constructed in 1930.

1.1. Gravity Collection System

The hydraulic model of DOU's wastewater collection and conveyance system includes gravity flow pipes and manholes. The key data for the pipes and manholes in the model include:

Pipes (links)	Manholes (nodes)
Pipe name	Manhole name
Upstream manhole	Ground surface elevation
Downstream manhole	Manhole invert elevation
Length	X coordinate
Cross section type	Y coordinate
Pipe diameter	
Upstream invert elevation	
Downstream invert elevation	

DOU provided these data in the configured sewer model.

The main interceptors serving the Aquia WWTP include the Route 1 Corridor Interceptor, the Staffordborough Interceptor, and the Austin Run Interceptor.

- Route 1 Interceptor was constructed in 1969 and ranges in size from 8 to 12 inches in diameter. The interceptor flows from north to south and discharges to the Aquia Creek Pumping Station.
- Staffordborough Interceptor ranges in diameter from 8 to 18 inches. This interceptor flows from west to east and discharges to the Aquia Creek Pumping Station.
- Austin Run Interceptor flows along Austin Run and discharges to the Austin Run Pumping Station. This interceptor ranges in size from 8 to 30 inches in diameter.

The primary interceptors serving the Little Falls Run WWTP include the Falls Run Interceptor and the Claiborne Run Interceptor.

- Falls Run Interceptor was initially built in 1975 and extended several times. It ranges from 8 to 18 inches in diameter. This interceptor discharges to the Falls Run Pumping Station which pumps flow to the Claiborne Run Interceptor.
- Claiborne Run Interceptor ranges from 8 to 36 inches in diameter. This interceptor discharges to the Claiborne Run Pumping Station which pumps directly to the Little Falls Run Wastewater Treatment Facility.

1.2. Pumping Stations and Force Mains

As a result of the hilly terrain in Stafford County, the sewer service area is composed of a number of sewersheds generally having higher elevations in the northwest and lower elevations to the southeast. There are a number of small pumping stations located throughout the system to pump flow between sewersheds. There are currently 83 pumping stations ranging in size from 18 to 6,520 gallons per minute (gpm). There are two major pumping stations upstream of the Aquia WWTP:

- Aquia Creek Pumping Station (2,000 gpm).
- Austin Run Pumping Station (4,020 gpm).

Two major pumping stations are located upstream of the Little Falls WWTP:

- Falls Run Pumping Station (6,520 gpm).
- Claiborne Run Pumping Station (5,600 gpm).

2.0. Level of Service Requirements

In general, the regulatory requirements for collection systems are becoming more stringent and there appears to be a trend toward a “zero tolerance” policy for sanitary sewer overflows. A sanitary sewer overflow (SSO) is the discharge of raw sewage from a municipal sanitary sewer system into basements, or out of manholes and pumping stations and onto city streets, playgrounds, and streams without any form of treatment. The USEPA and the VDEQ believe that inadequate management, operation and maintenance for sewage collection and conveyance systems pose a significant threat to receiving water quality and public health through the discharge of SSOs.

2.1. Capacity, Management, Operations and Maintenance

The USEPA is considering regulations and enforcement policies that will affect all municipal wastewater utilities by requiring all collection systems to be permitted through the National Pollutant Discharge Elimination System (NPDES) process. As part of this permitting process, utilities will be required to implement a Capacity, Management, Operations and Maintenance (CMOM) program.

In anticipation of the USEPA SSO policy, which may include a prohibition against sanitary sewer discharges, public utilities across the nation are working to ensure that their wastewater collection and conveyance systems can accommodate current and projected dry and wet weather flows without experiencing sanitary sewer overflows. The USEPA premise for the CMOM program is that when the permittee incorporates good business principles into its organization, the wastewater collection system will meet the intended performance standards and will ultimately have fewer SSOs. The CMOM program would place the burden of proof on the permittee to demonstrate that SSOs are being prevented through (1) use of pipes and pumping stations with adequate capacity, and (2) proper management, operations, and maintenance of the system. If the permittee cannot demonstrate that good business practices are being developed or in place when SSOs occur, the permittee could be deemed to be in violation of its NPDES permit.

The proposed CMOM program was developed, in part, to encourage all utilities to implement a proactive, rather than reactive, approach to wastewater collection system management, operations, and maintenance. According to both the USEPA and VDEQ, utilities with proper management, operation, and maintenance programs reduce the likelihood of SSOs, extend the life of their infrastructure, and provide better customer service through relatively steady rates and greater efficiency.

2.2. Performance Criteria

USEPA and VDEQ recognize that SSOs cannot be completely eliminated, and that sanitary sewer systems that are designed to not overflow when a given design storm occurs, may nonetheless experience wet weather induced overflows as the result of conditions other than the design storm. Therefore, as part of the NPDES permitting process, it is anticipated that USEPA and VDEQ will require local governments to certify that their sanitary sewer systems will not produce SSO events as a result of storm events equal to, or less than, a design storm of specified intensity and duration. The USEPA and VDEQ have yet to define the design storm criteria; however, the DOU collection and conveyance system has been analyzed for impacts associated with the 25-year peak inflow event.

Ideally, storm event data over at least 20 to 30 years are collected and the storms are ranked based on their effect on the sewer system (i.e., the amount of I/I caused in the system by the storm), rather than on individual storm characteristics (i.e., peak intensity, volume, and duration). The storms are commonly referred to as “peak inflow events” because the assigned return intervals more accurately refer to the ranking of the amount of I/I generated by the storm, rather than the actual size or characteristics of the storm. However, the significant effort needed to conduct an analysis of the impacts of historical storm events on the wastewater collection and conveyance system was not conducted in this study. Rather, inflow hydrographs were developed for storm events that occurred during the 2002 and 2003 flow monitoring period using hourly historical rainfall records. As described in Technical Memorandum 6, a 2-year storm event occurred on August 28, 2002 and was used for wet weather calibration of the hydraulic model and identification of the peak flow characteristics for a 2-year storm event.

An important aspect of RDI/I is its correlation to rainfall events and duration. Even within the same system, identical rainfall events may produce different wastewater flow reactions. It is difficult to predict a flow reaction from a large storm event based on data from a small event, as wastewater flows and rainfall intensity do not have a linear relationship.

3.0. Review of Hydraulic Model Tool

In general, DOU established the physical input data for the uncalibrated sewer model prior to initiation of the Master Plan. Sewer loads to be routed through the sewer system were established during the Master Plan.

3.1. Pipe Network Data and Connectivity

Pipes are conduits that transport flow through the sewer system either by gravity (i.e., gravity mains) or by the energy supplied from pumps (i.e., force mains). DOU staff performed quality control checks on the pipe network data and connectivity during model construction. In addition, the H2OMAP Sewer software performs a number of quality control checks on the system during model applications, including checks on the connectivity of the sewer system.

3.2. Sewer Loads

Sewer loads at manholes consist of sanitary base flow and flows resulting from rainfall events for the area tributary to the manhole. Groundwater infiltration is applied to the segments of pipe in the sanitary sewer system between the manholes. These sewer loads and infiltration represent estimates of the amount of wastewater flow that must be handled by the collection and conveyance system. The location in the hydraulic model for introducing the sewer load depends on the type of sewer load:

- Sanitary base flow - applied at manholes in the model of the sewer system.
- Groundwater infiltration (GWI) – applied to each segment of pipe in the sewer model.
- Rainfall-dependent I/I (RDI/I) - applied at manholes in the model of the sewer system.

DOU's GIS sewer layer and hydraulic model network of the sewer system were used as the basis for delineating the existing sewer service area. The service area boundary for future conditions (buildout) was based on the existing sewer service area, projected land use, sewershed boundaries (i.e., drainage basins, roadway and water features, etc.) and discussions with DOU and Planning Department staff regarding future development and policies. Once the sewer service areas were delineated, sewer loads were input by using a feature of the modeling software that compiles sewer loads and assigns the loads to the nearest manholes in the H2OMAP Sewer model using a polygon area surrounding the manhole. This approach results in an accurate allocation of sewer flows based on existing water meter billing data and future land use.

3.3. Model Calibration and Analysis

A functional, calibrated model was used to assess the performance of DOU's wastewater collection and conveyance system. The hydraulic model can be used to better understand and assess the capacity of the DOU's system by simulating and identifying hydraulic restrictions – surcharging pipes and overflowing manholes – within the system under specified flow conditions. The model was calibrated for 2002 and 2003 flows under dry and wet weather conditions. The wet weather calibration was based on the 2002 and 2003 flow monitoring data.

It is important to note that the model was calibrated using the storm events that occurred during the 2002 and 2003 flow monitoring period. Calibration is best when storm events with varying intensity, duration and volume are used. By using a variety of storm events, the inflow can be predicted for a range of storm events. The storm events that occurred during the 2002 and 2003 flow monitoring period had fairly short return periods (2-years or less) so the calibration was limited to these common storms. The characteristics and system response to more severe storm events may not be well predicted because the effects were extrapolated.

The hydraulic model will be a very valuable tool for DOU provided that the input files are maintained and updated as the collection and conveyance system expands and changes. This includes collecting additional data from flow monitoring to capture storms of varying characteristics. When used in conjunction with the other tools, such as GIS, SCADA, the model will serve as an integral part to the successful management and operation of the DOU collection and conveyance system.

A detailed discussion of model calibration is presented in Technical Memorandum 7 (*Development and Calibration of H2OMAP Sewer Hydraulic Model*).

4.0. Review of Sewer Loads

4.1. Introduction

The wet weather flow is used to assess the hydraulic capacity of the sewer system and is composed of three components:

- Sanitary base flow generated by homes, businesses, etc.,
- Infiltration due to normal groundwater levels (dry weather infiltration), and
- I/I due to rainfall and high groundwater levels (rainfall-dependent I/I)

The formula for calculating the sewer loads for wet weather conditions is as follows:

Peak Wet Weather Flow (PWWF) = Average Dry Weather Flow (ADWF) + Rainfall-Dependent I/I (RDI/I)

Where:

Peak Wet Weather Flow (PWWF) equals the peak hourly flow during wet weather conditions.

Average Dry Weather Flow (ADWF) is the average flow that occurs in sanitary sewers on a daily basis with no evident reaction to rainfall. The ADWF is composed of sanitary base flow and groundwater infiltration. Sanitary base flow equals the average daily water demand based on water duties multiplied by 80% which is an estimate of the customer water demand that is returned to the sanitary sewer. Groundwater infiltration (GWI) is an allowance that is added to the sanitary base flow (derived from sewage flow factors) to obtain the dry weather flow. GWI represents flow that is separate and distinguished from inflow resulting from storm events during wet weather conditions. The allowance used in this Master Plan for GWI is estimated to be 500 gpd/inch diameter-mile (gpdidm).

Rainfall-Dependent I/I consists of rainfall that enters the collection system through direct connections (roof leaders, manholes, etc.) and causes an almost immediate increase in wastewater flows. RDI/I data from an August 2002 storm event (2-year return interval) was used for sewer model calibration. For the August 28, 2002 storm event, peaking factors at various pumping stations ranged from 2.6 to 3.7 (i.e., peak hourly flows were 2.6 to 3.7 times greater than the average dry weather flow for that period). The weighted (based on number of upstream manholes) peaking factor for the overall sewer system was approximately 2.8 for the August 28, 2002 storm event.

Additional flow monitoring information is needed to accurately predict the response of the sewer system to larger storm events with varying characteristics (i.e., intensity, duration, and volume). To define the design flow conditions for the sewer system, the equation presented above was modified as follows:

Peak Wet Weather Flow (PWWF) = Average Dry Weather Flow (ADWF) x Peak Factor

The peak factor is equal to the PWWF/ADWF. In the H2OMAP Sewer model, the peak factor is multiplied by the sanitary base flow at each manhole in the sewer system and the GWI component (500 gpdidm) is subsequently added to the computed manhole flow as the flow is routed through the downstream sewer piping.

4.2. Sanitary Base Flows for Near-term Conditions

Sewer loads represent the average flows that are applied to the sewer system network from the contributing area. These demands are defined as the amount of flow that must be carried by the sewer system to satisfy the need. Manholes represent points in the sewer system where sewer loads are applied to the system. For the model of the existing system which was used for calibration, DOU provided the water demands based on customer billing data for 2001 which were reduced to 80% to obtain average sewer flows and applied to the nearest manholes. This approach results in an accurate allocation of current water demands to the nearest sewer manhole for sewer model calibration.

The near-term demands were provided by DOU and included in Appendix A of this technical memorandum. The near-term demands represent developments which could occur prior to 2010. The sewer loads were applied to the existing H2OMAP Sewer model to test the capabilities of the existing to handle the proposed flows. In some cases, piping was added to the existing model to reflect piping proposed for the new development. In addition, DOU identified that a few of the discharge points for the pumping station force mains were to be modified in the near future and these changes were incorporated.

4.3. Sanitary Base Flows for Buildout Conditions

Future sanitary base loads were projected using the estimated consumption method described in Technical Memorandum 2 (*Water Demands*). This method uses land use, customer class flow values, and flow ratios (peaking factors) to determine peak flow conditions. The general process for estimating the sanitary base flow at each manhole included:

- Establishing the base map of the service area. It should be noted that sewer service area was established and served as a boundary for calculating sewer loads.
- Obtain the land use areas and customer class assignments based on Land Use Plan.
- Calculate the sanitary base loads defined by land use customer classes as described in Technical Memorandum 2 (*Water Demands*).
- Overlay the map of land use customer classes and the manholes in the sewer model.
- Establish the area of influence for each manhole. Manhole areas of influence establish which loads will be assigned to which manholes. Generally, a line that (1) is perpendicular to the line connecting two manholes, and (2) intersects that line at its midpoint, will be used to determine the closest manhole to which the load can be assigned (loading polygon).
- Sum up the loads within each manhole's area of influence (loading polygon).
- Estimate peaking factors which are applied to the loads at each manhole.

This technique for assigning sewer loads to manholes in the model can easily accommodate changes in loading for land uses and reconfiguration of the model network.

4.4. Determination of Total Peak Design Flow

Design flow for a sewer is defined as the maximum flow rate that occurs under selected weather and growth conditions. Since a significant portion of the peak flows result from rainfall, the design storm flow that the sewer must convey is related to the probability of occurrence of a design storm event. Design flow for a selected rainfall event is the sum of the peak sanitary base flow, infiltration and inflow.

The design storm or storm recurrence interval is also the basis for prescribing a level of protection to the pipe capacity to carry the design flow. The selection of design storm becomes an integral component of the Capacity Assurance Plan (CAP) since it determines the threshold flows at which the sewer will be expected to surcharge and potentially overflow.

To establish the design storm for the sewer system, data from storm events that occurred during the flow monitoring period were analyzed to compute the R-value. The R-value is defined as the ratio of calculated RDI/I volume to the rainfall volume over the sewershed area, expressed as a percent. For example, an R-value of 0.10 indicates that 10% of the total monitored rainfall volume that fell over the sewershed made its way to into the sewer system as monitored RDI/I.

Rainfall data were reviewed for storm events that occurred during the period when flow monitoring data were collected:

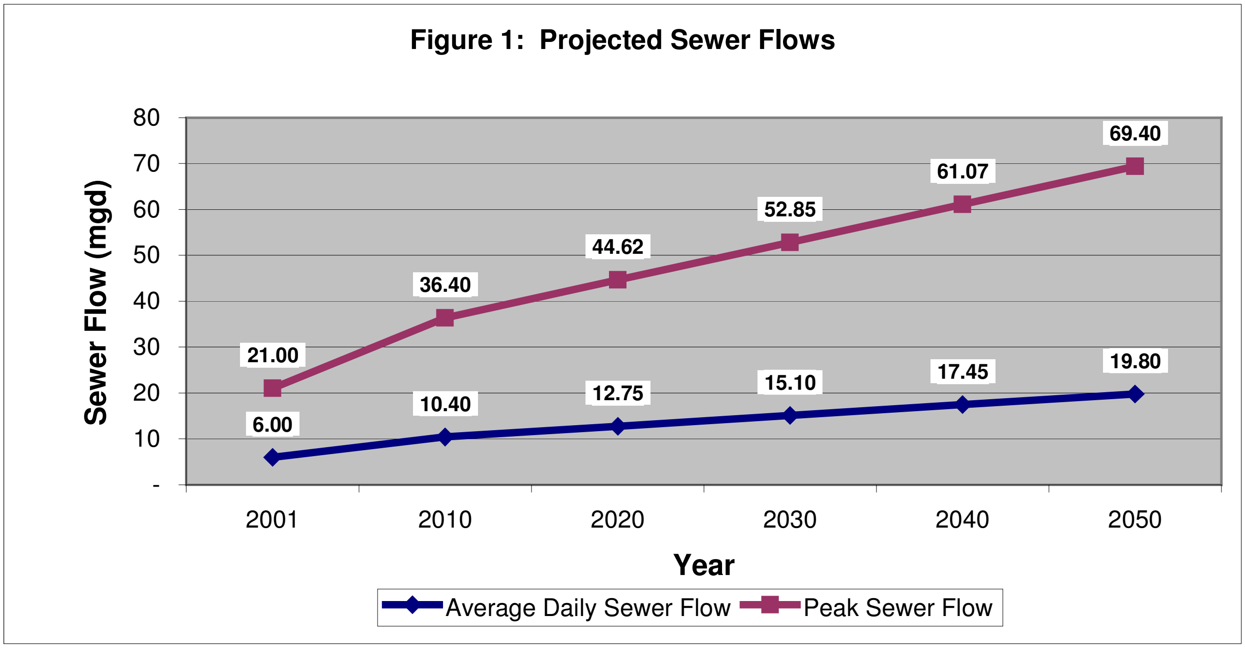
- August 28 – 30, 2002
- April 7 – 12, 2003
- April 18 – 19, 2003

The computation of R-values is presented in Appendix B of this Technical Memorandum.

Four rain gages in Stafford County were used for the analysis (Quantico, Garrisonville, K4HR, and Goldvein) as shown in the figure at the end of Technical Memorandum 6 *(Rainfall/Flow Monitoring Program*). Due to concerns related to rainfall data at the Goldvein gage and the K4HR gage for storm events during the flow monitoring period, data from the Goldvein and K4HR gage were used to generate a representative storm event for each of the three periods of rainfall. Using the inflow data from flow monitoring for the storm events in combination with the volume of rainfall that occurred during the period, the average R-values based on the Quantico, Garrisonville, K4HR/Goldvein gages were estimated to be 1%, 1.3%, and 0.91%, respectively.

The system-wide RDI/I was computed for various storm events using the 24-hour rainfall totals from IDF curves and the average R-values. Combining the system-wide RDI/I with the dry weather flow yielded an estimated peak wet weather flow for the overall future sewer system for various storm events (see figure in Appendix B). In addition, the system-wide sewer flow associated with various peaking factors can be computed by multiplying the average dry weather flow at buildout times the peaking factors. Based on the results of the August 2002 storm event, industry guidelines, and anticipated regulatory requirements, a peak factor of 3.5 is used to derive the peak wet weather flow for a storm event with an estimated 25-year recurrence interval. Appendix B of this technical memorandum shows the calculation of R-values and the precipitation frequency estimate for a 25-year 24-hour storm event (5.9 inches).

Average daily sewer flows are expected to increase from approximately 6.0 mgd (2001) to roughly 19.8 mgd under buildout (2050) conditions. During the same period, the maximum day demands are expected to increase from approximately 21 mgd (2001) to 69.4 mgd at buildout (2050) based on a peaking factor of 3.5 times the average daily flow. The sewer flow projections are shown in Figure 1.



5.0. Review of Sewer Planning and Design Criteria

A sanitary sewer collection system has basically two main functions: (1) to convey the design peak discharge, and (2) to transport solids so that deposits are kept to a minimum. It is imperative, therefore, that the sanitary sewer has adequate capacity for the peak flow and that it functions at minimum flows without excessive maintenance and generation of odors.

A comparative review of DOU's Planning and Design criteria for sewer systems was performed to identify whether the sewer system criteria proposed for use in the Water and Sewer Master Plan project are reasonable. The planning and design criteria will be used to evaluate the sewer system and to plan future improvements, upgrades, and expansions of facilities.

While national organizations provide some guidelines and many states regulate certain performance criteria, design criteria are often left to the discretion of the utility. The planning and design criteria proposed for use in DOU's Water and Sewer Master Plan project were compared with the criteria used by similar utilities in the region (e.g., location, estimated population served, growth rate, customer demographic, etc.).

This information was reviewed with DOU to identify which planning and design criteria should be modified to reflect recent or anticipated future changes and to document policy decisions regarding application of the criteria. Understanding the potential impacts that revising the planning and design criteria may have on the existing and proposed capital improvements is essential. Additional studies (e.g., flow monitoring, historic flow data, etc.) may be needed in the future to more clearly define the desired modifications to the criteria.

5.1. Evaluation of Planning and Design Criteria

Wastewater planning and design criteria used by DOU was reviewed in conjunction with the Virginia Department of Health (VDH) Sewage Collection and Treatment (SCAT) regulations and the criteria of select Virginia communities to evaluate and update, as necessary, DOU's sewer criteria. Table 2 presents a comparison of the sanitary sewer design criteria for various collection systems throughout the state of Virginia as well as the VDH SCAT regulations in comparison to DOU's requirements.

Table 2. Sewer design criteria

Reference	"n" value	Minimum Velocity (fps)	Maximum Velocity (fps)	Minimum Depth of Cover (ft)	Maximum Depth of Cover (ft)	Pipe Flowing ___ % Full
Stafford County Requirements	***0.013***	***2.25***	***15***	***3.0***	***20***	***80%***
Virginia SCAT Regulations	0.014	2.0	15 [1]	sufficient depth to prevent ice formation	Not Defined.	100%
Chesterfield County	0.012	2.25	15 [1]	3.5 (6' min. required under existing roadways)	Not Defined.	100%
Fauquier County WSA	0.013	2.0	10 [1]	3.5	18 (or special pipe material required)	80%
Hanover County	0.013	2.0	15 [1]	4.0 (6' min. required under existing roadways)	Not Defined.	100%
Henrico County	0.013 (8"-21") 0.012 (>24")	2.25	15 [1]	5.5 (ROW) 3.5' (easement)	18 (or special pipe material required)	100% or 50%
Prince William County	0.013	2.25	10 [1]	5.0 (street) 3.5 (ductile iron)	18 (or special pipe material required)	80%

Notes:

1. When maximum velocities are exceeded additional design criteria must be met.

Based on a review of the information presented in Table 2, it is evident that the sanitary sewer design criteria currently utilized by DOU is appropriate, as much of the criteria is identical between communities. Where differences do occur, as is the case with the Manning's "n" value, they are slight. Furthermore, DOU's design criteria meet or exceed the VDH SCAT regulations with the exception of the Manning's "n" value. However, although the required "n" value in the SCAT regulations is more conservative than that employed by DOU, a more conservative minimum velocity is required by DOU. Therefore, as velocity is a function of Manning's "n", Stafford County's requirements were determined to be adequate and determined to meet the overall requirements of the VDH SCAT regulations.

The sewer planning and design criteria used in this Master Plan include the following:

"n" value	0.013 for all pipe materials
Minimum Velocity	2.25 ft/sec
Maximum Velocity	15 ft/sec
Minimum Depth of Cover	3 feet
Maximum Depth of Cover	20 feet

5.2. Analysis Curves and Design Curves

The H2OMAP Sewer model includes analysis and design criteria curves which are effective and efficient tools that can be used under steady-state conditions for evaluating the capabilities of the existing system and sizing improvements to the sewer system.

5.2.1. Analysis Criteria Curve

An analysis criteria curve has been developed for this study to define the "threshold" values at which point capacity enhancement measures for pipelines within the sanitary sewer system should be evaluated. There are no established requirements or guidelines for q/Q ratios. Selection of the q/Q ratios and the associated range of pipeline sizes are based on best professional judgement taking into consideration the following:

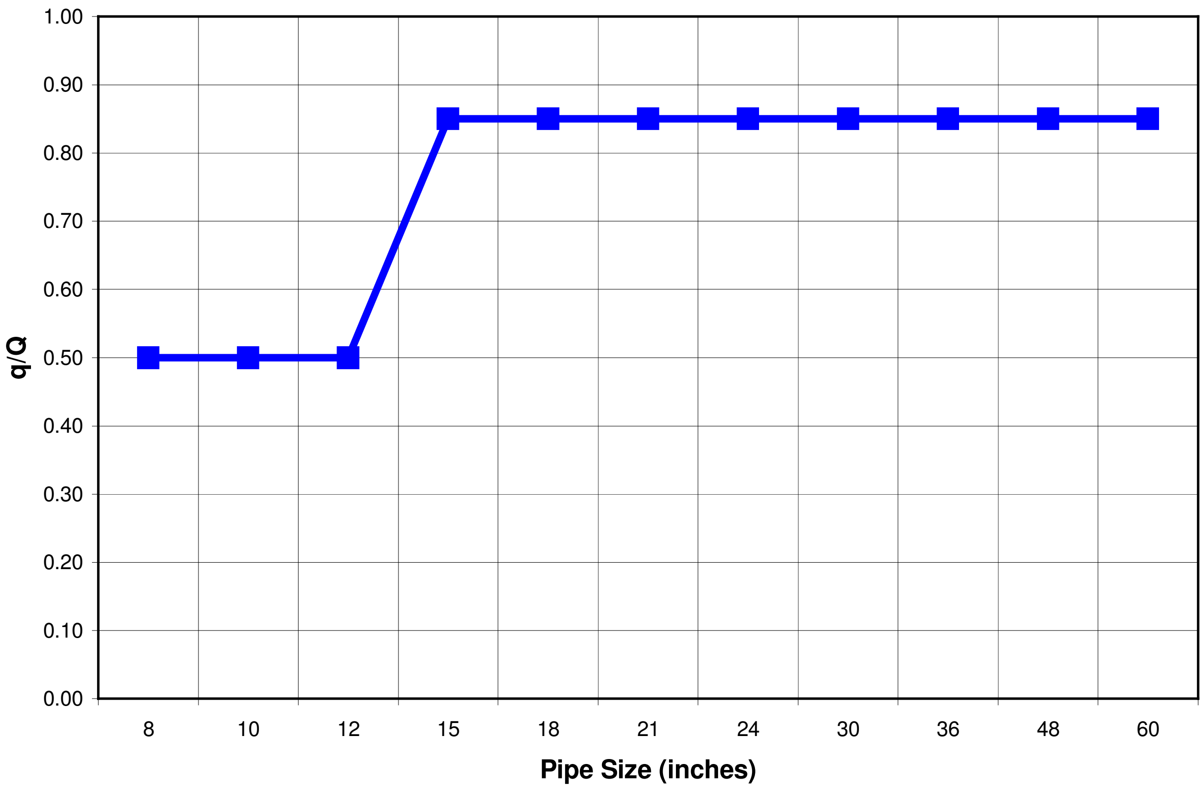
- Potential delays associated with implementation of future improvements (e.g., planning, siting, design, and construction).
- Risk of sanitary sewer system overflows.
- Excess capacity in sanitary sewer pipelines resulting in higher maintenance and possible odors.
- Rate of development (i.e., timing for additional future improvements).
- Potential for additional future development.

Based on these considerations, the values shown in Table 3 and Figure 2 are proposed for the initial analysis criteria curve proposed for use in this study.

Table 3. Analysis criteria curve

Pipeline Diameter	q/Q Ratio
8-inch through 12-inch	0.50
15-inch and up	0.85

Figure 2 - Analysis Criteria Curve



The American Society of Civil Engineers *(Gravity Sanitary Sewer Design and Construction,* 1982) identified the following:

> *"It is customary to design sanitary sewers with some reserve capacity. Generally, sanitary sewers through 15 inches in diameter are designed for flow half full. Larger sanitary sewers are designed to flow three quarters full."*

The initial partial flow-to-full flow ratios used to develop the analysis criteria curve shown in Figure 2 were less conservative for the large diameter sewer pipelines (greater than 15 inches in diameter). The q/Q ratio of 0.85 (d/D ratio of 0.75) for the large diameter pipelines reflects the desire to maximize flow in the existing interceptor sewers while maintaining some reserve capacity and reflects the uncertainty in the spatial distribution of sewer loads served by the smaller piping in the sewer system. By applying relatively conservative q/Q ratios for the analysis curve, pipelines will be identified prior to reaching full capacity and thus reduce the likelihood of surcharge and/or overflow conditions. It should be noted that existing pipelines that exceeded the design criteria and were less than full through buildout conditions (q/Q less than 1.0) were not recommended for replacement. Rather, these pipelines were flagged for future investigation and possible flow monitoring during the planning period.

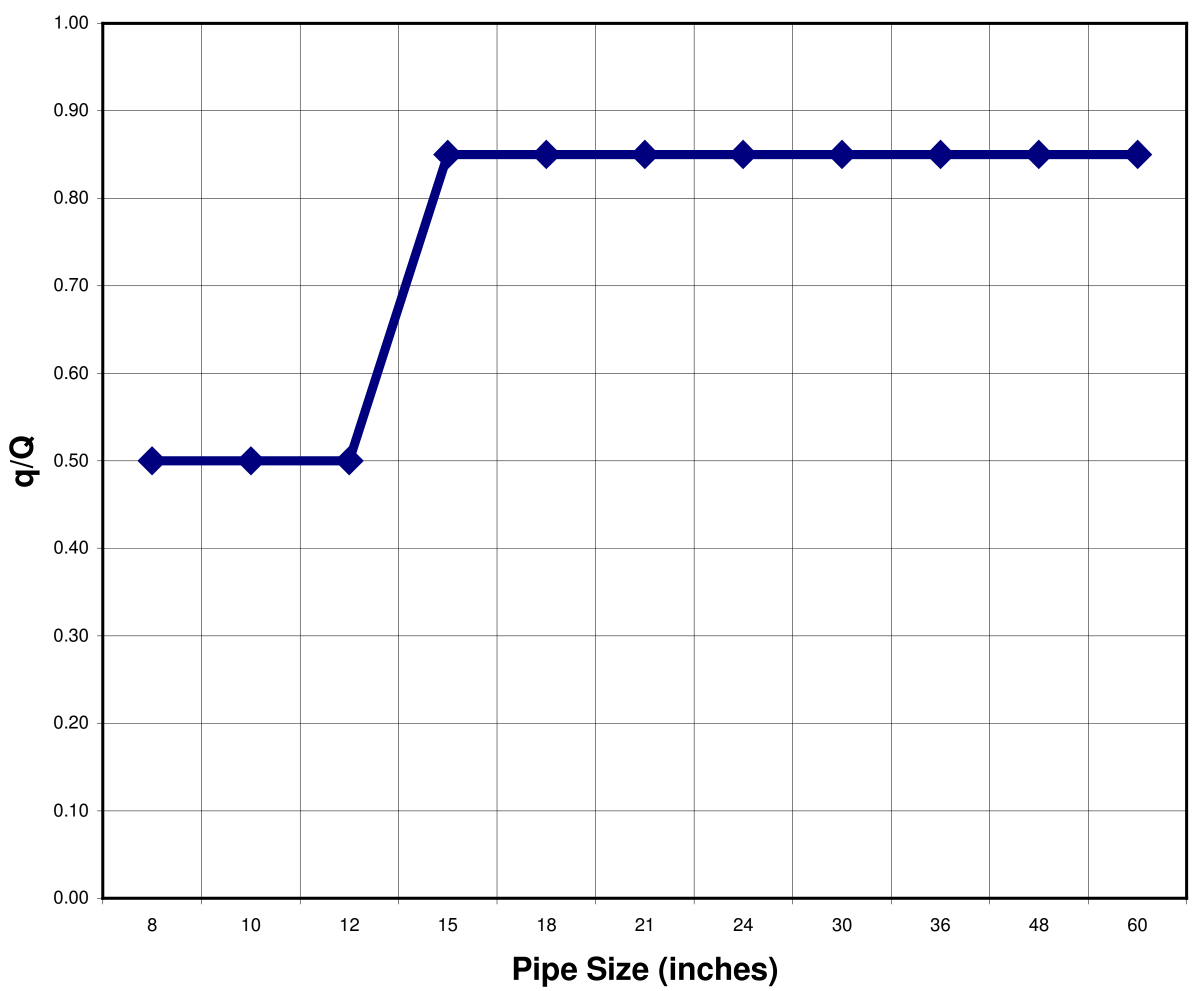
5.2.2. Design Criteria Curve

The design criteria curve is used for designing the relief or replacement pipelines when the capacity of the existing pipelines has been exceeded as defined by the analysis criteria curve. In general, the design criteria curve generally reflects the desire to limit the possibility of requiring additional improvements in the near-term planning period. The initial design criteria curve values proposed for use in this study are shown in Table 4 and are plotted in Figure 3.

Table 4. Design criteria curve

Pipeline Diameter	q/Q Ratio
8-inch through 12-inch	0.50
15-inch and up	0.85

Figure 3 - Design Criteria Curve



To utilize the analysis and design criteria curves, hydraulic modeling under steady-state conditions was conducted to analyze the performance of the sanitary system. A steady state simulation is a single, instantaneous "snap-shot" of a sewer collection system. In H2OMAP Sewer, there are two types of steady state simulations:

- Steady state analysis – A standard hydraulic simulation that performs a "snap-shot" analysis of the collection system and determines the q/Q ratio, flow, velocity, and excess capacity of each pipe. This simulation also determines surcharge conditions.
- Steady state design – The same as steady-state analysis, except q/Q and velocity are applied to the analysis and replacement or parallel facilities are recommended.

For steady-state modeling, manhole loading patterns and controls (e.g., pumps, etc.) used in extended period simulations are not considered because the simulation represents a "snap-shot" analysis. Manhole loads under steady-state simulations can be either unpeaked or peaked as follows:

- Unpeakable flow – The corresponding load at each manhole is modeled as a direct flow into the sewer system (i.e., $Q_{unpeaked} = Q_{base}$).
- Peakable flow – For this master plan, a global peaking factor of 3.5 is used to generate peak wet weather flows.

For a given model run, the model used the information from the analysis criteria curve to identify pipelines which failed the capacity guidelines defined by the q/Q or d/D ratios. After identifying system capacity limitations, the model performs iterations using the design criteria curve and design cost curves for new pipes (i.e., different pipe materials, roughness coefficients, etc.) to identify the replacement or relief pipe that satisfies the conditions defined in the analysis and design criteria curves. Modeling runs using alternative analysis and design criteria curves can be performed to assess the impact the criteria have on the need for and timing of future improvements.

5.3. Pumping Station Controls

Pumping stations in the H2OMAP Sewer model can be evaluated using a number of different control settings:

- Control Method - The control method specifies the criteria that dictates the ON and OFF settings of a pump.
- By Volume - The pump operates depending on the volume of sewage in the wetwell. The user should provide the on/off volumes.
- By Level - The pump may turn on or off based on water level in the wetwell. The on/off levels need to be specified for this control option.
- By Discharge - This control method dictates a mechanism in which a user supplied pattern of targeted pump discharged flow is maintained. H2OMap Sewer adjusts speed of the pump to insure that the desired amount of flow is pumped. This option is available for one-point and exponential three-point pumps only, and is not valid for lift (pump) stations with parallel pumps. The actual pumped flow could differ from the desired flow if the minimum and the maximum water levels set for the wetwell are violated during the simulation duration. In other words, the pump may turn off if the water level in the wetwell falls below the minimum wetwell level allowed in spite of the desired flow determined based on the pattern. Likewise, the pump may turn on if the water level in the wetwell exceeds the maximum allowed water level even if the targeted flow is zero.
- By Inflow - Under this control alternative, the level of water in the wetwell remains constant during the simulation duration. The pump turns on if there is inflow to the wetwell. The discharged flow is equal to the inflow to the wetwell. This control alternative is ideally suited for wetwells that do not have enough storage volume. This option is available for one-point and exponential three-point pumps only, and is not valid for lift (pump) stations with parallel pumps.
- By Time - The "By Time" control option offers the user the flexibility to turn the pump on/off at any time of a day. The model accepts the operational schedule in the form of a speed pattern. The pump turns off if the speed setting is zero, and turns on otherwise.

For this Master Plan, steady-state analyses were conducted with pumps controlled by either the "By Discharge" or "By Inflow" setting. Under the buildout, conditions, several of the larger pumping stations needed to be changed to "By Inflow" to reflect flow conditions which exceeded the current capacity of the existing stations.

5.4. Summary

The design criteria curves presented in this section were used to evaluate the capabilities of the sanitary sewer system under steady state conditions. The analysis and design criteria curves presented in this memorandum are proposed as the basis for identifying the sanitary sewer system deficiencies and future improvements. Modeling runs using alternative analysis and design criteria curves, as well as peaking factors, were performed to assess the impact the criteria have on the need for and timing of future improvements. Decisions regarding the need to implement the identified future improvements will require consideration on a case-by-case basis taking into consideration a number of factors, such as remaining development potential.

6.0. System Capacity Analysis

The system capacity analysis was conducted using the calibrated hydraulic model to simulate the system's response to wet weather flow conditions and to identify hydraulic capacity restrictions in the DOU wastewater collection system.

6.1. Classification of Hydraulic Restrictions

Pipes were identified as being "surcharged" if they were predicted to be flowing at 100 percent full during some point of the modeling simulation. Once the modeling simulations were completed and the results analyzed, the surcharging pipes and overflowing manholes were identified. The locations of hydraulic restrictions were prioritized into one of two possible categories:

- Major hydraulic restriction areas
- Minor hydraulic restriction areas

Major hydraulic restriction areas were identified as those areas where a large number of gravity pipes were predicted to surcharge and multiple manholes were predicted to overflow (i.e., numerous segments over 100 percent full). Minor hydraulic restriction areas were identified as those areas where pipes were less than 100 percent full, but exceeded the analysis and design criteria curve values (i.e., q/Q greater than 0.5 for pipes 12 inches and smaller, and q/Q greater than 0.85 for pipes 15 inches and larger). Improvements were proposed for those areas with major hydraulic restrictions, and pipe segments were flagged in yellow on the Wastewater Improvements Map at the end of this Master Plan. In general, minor hydraulic restrictions were caused by small, localized hydraulic characteristics of the system.

6.2. Modeling Scenarios

The calibrated model was used to identify major hydraulic restrictions – surcharging pipes and overflowing manholes – within the collection system under a variety of dry and wet weather flow conditions. The hydraulic modeling effort was conducted for two flow scenarios:

- Wet weather "Near-term Conditions" (2010)
- Wet weather "Future Conditions" (buildout)

The wet weather scenarios for "Near-term Conditions" and "Future Conditions" were designed to simulate system responses based on 2003 and buildout dry weather wastewater flow projections in addition to flow volumes associated with the 25-year inflow event. It is important to note that the 25-year storm event routed through the model network under the wet weather flow scenarios was larger than those storm events captured during the 2002 and 2003 flow monitoring program which were used for model calibration. Due to the limited wet weather data to calibrate the model, the collection system's actual response to these larger storms may vary from the model's predictions.

The model network used for the "Near-term Conditions" scenario included the 4-inch and larger pipes and pumping stations throughout the DOU service area. The physical network used for the "Near-term Conditions" was identical to the model network for the existing system with the near-term flows included and some piping added to unsewered area which will be developed in the near-term.

6.3. Modeling Results

The capital improvements recommended based on the modeling results are described in the following section.

7.0. Recommended Plan of Action

Utilities are utilizing a variety of strategies and tools to assess the current performance of their collection systems and identify hydraulic capacity problems that need to be addressed. Owners and operators are using integrated approaches to address their infrastructure problems through a variety of technologies including, but not limited to:

- Hydraulic sewer modeling
- Supervisory Control and Data Acquisition (SCADA) systems
- Temporary and long-term flow monitoring programs
- Geographic Information Systems (GIS)

Capital improvement program (CIP) recommendations were developed for the areas with hydraulic restrictions and deficiencies. The CIP recommendations were developed to address capacity and condition deficiencies identified during the assessment of the wastewater collection and conveyance system.

For each of the hydraulic restriction areas identified, multiple alternatives were investigated. The purpose of the analysis and discussion within this technical memorandum is to offer a few key alternatives to address the problem areas. For each of the areas with hydraulic restrictions, an alternatives analysis was conducted to evaluate scenarios to correct the problem under current and future conditions. Simulation of the 25-year storm event with the buildout wastewater flow projections was used to determine if the recommended alternatives effectively relieve the hydraulic restrictions. The hydraulic restriction and course of action to correct the restriction are outlined below.

A map showing the proposed improvements and the summary of the cost and timing of improvements are included in the pockets at the end of this Master Plan.

7.1. Emergency Generators

The USEPA's SSO policy will prohibit sanitary sewer discharges and require DOU to certify that their wastewater collection and conveyance system can accommodate current and projected dry and wet weather flows without experiencing sanitary sewer overflows. The improvements described below address the capacity of the wastewater collection system. However, the reliability of wastewater pumping stations should be considered to prevent SSOs during power outages. Given the frequency of severe thunderstorm, the threat of hurricanes and the likelihood that other storms may result in power outages, many utilities, including DOU, are making provisions for emergency power generators at their wastewater pumping stations to reduce the risk of SSOs during power outages.

7.2. Widewater Alternatives

The Widewater development is proposed in the northeastern portion of the County north of Aquia Creek. For this Master Plan, three scenarios for Widewater were considered using flow information provided by DOU:

- No sewer flow from Widewater assuming that Widewater would construct its own facilities, if any. The existing 8-inch and 10-inch gravity mains along Telegraph Road are approximately 10% to 15% full under buildout conditions. For this option, there is no cost for improvements to the existing sewer system downstream of the Telegraph Road due to development of Widewater.

- Conveyance of 400 gpm from Widewater to the existing 8-inch and 10-inch gravity mains on Telegraph Road. The sewer improvements downstream of the Widewater connection along Telegraph Road include replacing the existing 8-inch and 10-inch gravity mains with a 12-inch main ($298,000) and a 15-inch main ($350,000), and expansion of the Aquia Creek PS ($201,000). The existing 18-inch force main from the Aquia Creek PS to the Aquia WWTP should have sufficient capacity at buildout to handle the additional 400 gpm from Widewater.

- Conveyance of 6,280 gpm from Widewater to the existing 8-inch and 10-inch gravity mains on Telegraph Road. The sewer improvements downstream of the Widewater connection along Telegraph Road include replacing the existing 8-inch and 10-inch gravity mains with a 36-inch main ($930,000), expansion of the Aquia Creek PS ($2,500,000), and construction of a 24-inch force main from Aquia Creek PS to Aquia WWTP ($1,500,000).

7.3. Garrisonville PS and Interceptor along Austin Run

An expansion of the Garrisonville PS from 3.5 mgd to 5.6 mgd at buildout is proposed in this section (A-202). For the buildout condition, it was concluded from modeling that the capacity (assuming full flow) of the interceptor downstream of the force main serving the Garrisonville PS could accept a maximum of approximately 700 gpm (1 mgd) of flow from the area west of the Garrisonville PS (i.e., vicinity of Lake Arrowhead). Flows in excess of roughly 1 mgd would require upsizing the existing interceptors along Austin Run and Blossom Wood Court at a cost of roughly $1.6 million. Under buildout conditions, the existing interceptor downstream of the force main serving Garrisonville PS flows between 55% and 80% full. It is anticipated that flows from the area west of the Garrisonville PS would not be conveyed to the Garrisonville PS until 2025 or beyond.

7.4. Sewer Service Area Boundary

DOU's GIS sewer layer and hydraulic model network of the sewer system were used as the basis for delineating the existing sewer service area. The service area boundary for future conditions (buildout) was based on the existing sewer service area, projected land use, sewershed boundaries (i.e., drainage basins, roadway and water features, etc.) and discussions with DOU and Planning Department staff regarding future development and policies. The boundary represents a "wall" and areas outside the service area envelope were not included in this Master Plan.

7.5. Planning Periods

Sewer loads for two planning periods were evaluated in this Master Plan: near-term (2010) and buildout (2050). The near-term demands were based on 2001 water billing data reduced to sewer loads and applied to the nearest manhole plus the proposed developments included in Appendix A. The buildout flows were projected based on flows generated by the land use tributary to each manhole.

7.6. Aquia Wastewater Treatment Facility Service Area

7.6.1. Aquia WWTP - Gravity Mains

A-3: Construct 18-inch gravity main from Courthouse Road near Cedar Lane to Rocky Run Interceptor (3,519 feet)

This project includes design and construction of an 18-inch gravity main from Courthouse Road near Cedar Lane to Rocky Run Interceptor (3,519 feet). The purpose of the project is to serve customers in the vicinity of Embrey Mill and convey flows from the Upper Accokeek PS and the Route 630 PS. The timing for construction of this project is concurrent with construction of the Rocky Run Interceptor and the force main from the Upper Accokeek PS.

Priority	*2 – Near-term*
Design	*2007*
Construct	*2008*
Design Flow (Upstream/Downstream)	*2,917 gpm/3,228 gpm*
Total Project Cost	*$775,000*
Prior Spending	*$0*
Costs in this Plan Period	*$775,000*
Pro Rata Area	*Austin Run & Accokeek Creek*

A-4: Construct 12-inch gravity main along Accokeek Creek from location downstream of Rowser PS (3,121 feet)

This project includes design and construction of a 12-inch gravity main along Accokeek Creek from location downstream of Rowser PS (3,121 feet). The purpose of the project is to serve future customers in the area tributary to Accokeek Creek between I-95 and Jefferson Davis Highway. The timing for construction of this project is dependent on the timing of future flows in the area and is concurrent with construction of the Lower Accokeek PS and interceptor.

Priority	*3 - Buildout*
Design	*2021*
Construct	*2022*
Design Flow (Upstream/Downstream)	*234 gpm/484 gpm*
Total Project Cost	*$432,000*
Prior Spending	*$0*
Costs in this Plan Period	*$432,000*
Pro Rata Area	*Accokeek Creek*

A-5: Replace 8-inch with 12-inch gravity main from Venture Road to Wyche Road PS (1,190 feet)

This project includes replacement of the existing 8-inch with 12-inch gravity main from Venture Road to Wyche Road PS (1,190 feet). The purpose of the project is to serve future customers in the area tributary to Accokeek Creek between I-95 and Jefferson Davis Highway. The existing 8-inch main is roughly 10% full under near-term conditions and 75% to 100% full under buildout conditions. The timing for construction of this project is dependent on the timing of future flows in the vicinity of Venture Road and is concurrent with construction of the Lower Accokeek PS and interceptor.

Priority	*3 - Buildout*
Design	*2021*
Construct	*2022*

Design Flow (Upstream/Downstream)	*809 gpm/838 gpm*
Total Project Cost	*$231,000*
Prior Spending	*$0*
Costs in this Plan Period	*$231,000*
Pro Rata Area	*Accokeek Creek*

A-14: Replace 8-inch and 10-inch with 18-inch gravity main along Jefferson Davis Highway from Carnaby Street to Austin Run PS (3,017 feet)

This project includes replacement of the existing 8-inch and 10-inch with 18-inch gravity main along Jefferson Davis Highway from Carnaby Street to Austin Run PS (3,017 feet). The purpose of the project is to serve future customers along the Jefferson Davis Highway corridor south of Aquia WWTP. The timing for construction of this project is dependent on the timing of flows in the area between Jefferson Davis Highway and Olde Concord Road and is concurrent with the proposed 18-inch interceptor (A-30).

Priority	*2 – Near-term*
Design	*2009*
Construct	*2010*
Design Flow (Upstream/Downstream)	*3,501 gpm/3,667 gpm*
Total Project Cost	*$792,000*
Prior Spending	*$0*
Costs in this Plan Period	*$792,000*
Pro Rata Area	*Austin Run & Accokeek Creek*

A-16: Replace 8-inch with 12-inch gravity main from vicinity of Nina Cove to Jefferson Davis Highway (1,390 feet)

This project includes replacement of the existing 8-inch with 12-inch gravity main from vicinity of Nina Cove to Jefferson Davis Highway (1,390 feet). The purpose of the project is to increase the conveyance capacity of the existing 8-inch gravity main. The existing 8-inch main is roughly 50% full under near-term conditions and 70% to 100% full under buildout conditions. Prior to replacing the existing gravity main, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing gravity main.

Priority	*7 – Flow monitoring*
Design	*2014*
Construct	*2014*
Design Flow (Upstream/Downstream)	*234 gpm/302 gpm*
Total Project Cost	*$270,000*
Prior Spending	*$0*
Costs in this Plan Period	*$270,000*
Pro Rata Area	*Austin Run*

A-18: Replace 24-inch with 36-inch gravity main along Austin Run from Whitsons Run to Austin Run PS (3,548 feet)

This project includes replacement of the existing 24-inch with 36-inch gravity main along Austin Run from Whitsons Run to Austin Run PS (3,548 feet). The purpose of the project is to increase the capacity of this critical interceptor which conveys flow from the interceptors along Austin and Whitsons Run under I-95 to the Austin Run PS. The existing 24-inch main is roughly 75% to 80% full under near-term flow conditions and exceeds full flow under buildout conditions. This project serves a large area and a major source of flow impacting the timing for replacing the existing gravity main is the quantity of flow through the Camp Barrett PS (Quantico Marine Corps Base). Delays in the quantity of flow from

Quantico Marine Corps Base could delay the construction of this project. Due to the importance of maintaining adequate conveyance capacity in this interceptor, replacement is recommended in the 2012-2013 timeframe.

Priority	*3 - Buildout*
Design	*2012*
Construct	*2013*
Design Flow (Upstream/Downstream)	*11,718 gpm/21,002 gpm*
Total Project Cost	*$2,366,000*
Prior Spending	*$0*
Costs in this Plan Period	*$2,366,000*
Pro Rata Area	*Austin Run*

A-23: Replace 10-inch with 12-inch gravity main along unnamed tributary to Aquia Creek and Choptank Road from Garrisonville Road to Huckstep Avenue (2,946 feet)

This project includes replacement of the existing 10-inch with 12-inch gravity main along unnamed tributary to Aquia Creek and Choptank Road from Garrisonville Road to Huckstep Avenue (2,946 feet). The purpose of the project is to increase conveyance capacity of the existing 10-inch gravity main. The existing 10-inch main is roughly 55% to 75% full under near-term (2010) flow conditions and exceeds full flow under buildout conditions. Prior to replacing the existing gravity main, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing gravity main.

Priority	*7 – Flow monitoring*
Design	*2014*
Construct	*2014*
Design Flow (Upstream/Downstream)	*473 gpm/726 gpm*
Total Project Cost	*$730,000*
Prior Spending	*$0*
Costs in this Plan Period	*$730,000*
Pro Rata Area	*Austin Run*

A-27: Construct 8-inch gravity main along South Austin Run from Mine Road to PS on September Lane (4,213 feet)

This project includes design and construction of an 8-inch gravity main along South Austin Run from Mine Road to PS on September Lane (4,213 feet). The purpose of the project is to serve future customers along South Austin Run and eliminate the pumping station along September Lane. The timing for construction of this project is dependent on the timing of flows in this area and should be implemented prior to exceeding the capacity of the pumping station along September Lane.

Priority	*2 – Near-term*
Design	*2010*
Construct	*2011*
Design Flow (Upstream/Downstream)	*188 gpm/354 gpm*
Total Project Cost	*$573,000*
Prior Spending	*$0*
Costs in this Plan Period	*$573,000*
Pro Rata Area	*None*

A-28: Construct 8-inch gravity main along unnamed tributary to Whitsons Run from vicinity of Craftsman Court to Highpointe Boulevard and Mine Road (1,417 feet)

This project includes design and construction of an 8-inch gravity main along unnamed tributary to Whitsons Run from vicinity of Craftsman Court to Highpointe Boulevard and Mine Road (1,417 feet). The purpose of the project is to serve future customers in the Highpointe area and convey flows to the Whitsons Run Interceptor thereby making capacity available in the existing interceptor along Garrisonville Road.

Priority	*2 – Near-term*
Design	*2006*
Construct	*2007*
Design Flow (Upstream/Downstream)	*120 gpm/150 gpm*
Total Project Cost	*$193,000*
Prior Spending	*$0*
Costs in this Plan Period	*$193,000*
Pro Rata Area	*None*

A-29: Replace 8-inch with 15-inch gravity main along Greenspring Drive from Whitsons Run Drive to Stafford Glen Court (350 feet)

This project includes replacement of the existing 8-inch with 15-inch gravity main along Greenspring Drive from Whitsons Run Drive to Stafford Glen Court (350 feet). The purpose of the project is to increase the capacity of the existing 8-inch gravity main to meet near-term demand conditions.

Priority	*2 – Near-term*
Design	*2008*
Construct	*2009*
Design Flow	*172 gpm*
Total Project Cost	*$101,000*
Prior Spending	*$0*
Costs in this Plan Period	*$101,000*
Pro Rata Area	*Austin Run*

A-30: Construct 18-inch gravity main along unnamed tributary from Olde Concord Road to interceptor along Jefferson Davis Highway near Carnaby Street (5,833 feet)

This project includes design and construction of an 18-inch gravity main along unnamed tributary from Olde Concord Road to interceptor along Jefferson Davis Highway near Carnaby Street (5,833 feet). The purpose of the project is to serve future customers east of Jefferson Davis Highway near Somerset Lane. The timing for construction of this project is dependent on the timing of flows in this area which is anticipated to be in the near-term (through 2010). The force main from the Courthouse PS should be connected to this 18-inch gravity main in the future to alleviate future capacity issues associated with the gravity sewers downstream of the existing force main serving the Courthouse PS. In the future, this project will convey flows from the force main serving the Lower Accokeek PS.

Priority	*2 – Near-term*
Design	*2009*
Construct	*2010*
Design Flow (Upstream/Downstream)	*2,838 gpm/2,952 gpm*
Total Project Cost	*$1,056,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,056,000*

Pro Rata Area	*Austin Run & Accokeek Creek*

A-31: Construct 12-inch gravity main along unnamed tributary to Accokeek Creek from Wyche Road PS to interceptor along Accokeek Creek (1,638 feet)

This project includes design and construction of a 12-inch gravity main along unnamed tributary to Accokeek Creek from Wyche Road PS to interceptor along Accokeek Creek (1,638 feet). The purpose of the project is to eliminate the Wyche Road PS and serve future customers downstream of the Wyche Road PS. The timing for construction of this project is dependent on the timing for construction of the Lower Accokeek PS and interceptor.

Priority	*3 - Buildout*
Design	*2021*
Construct	*2022*
Design Flow (Upstream/Downstream)	*841 gpm/849 gpm*
Total Project Cost	*$240,000*
Prior Spending	*$0*
Costs in this Plan Period	*$240,000*
Pro Rata Area	*Accokeek Creek*

A-32: Construct 10-inch gravity main from Rowser PS to interceptor along Accokeek Creek (626 feet)

This project includes design and construction of a 10-inch gravity main from Rowser PS to interceptor along Accokeek Creek (626 feet). The purpose of the project is to eliminate the Rowser PS and serve future customers downstream of the Rowser PS. The timing for construction of this project is dependent on the timing for construction of the Lower Accokeek PS and interceptor.

Priority	*3 - Buildout*
Design	*2021*
Construct	*2022*
Design Flow	*139 gpm*
Total Project Cost	*$96,000*
Prior Spending	*$0*
Costs in this Plan Period	*$96,000*
Pro Rata Area	*Accokeek Creek*

A-33: Construct 18-inch gravity main along Accokeek Creek from vicinity of Jumping Branch Road to Lower Accokeek PS (4,737 feet)

This project includes design and construction of an 18-inch gravity main along Accokeek Creek from vicinity of Jumping Branch Road to Lower Accokeek PS (4,737 feet). The purpose of the project is to serve future customers in the vicinity of the Lower Accokeek PS and convey flows from the Wyche Road PS and the Rowser PS. The timing for construction of this project is dependent on the timing for construction of the Lower Accokeek PS.

Priority	*3 - Buildout*
Design	*2021*
Construct	*2022*
Design Flow (Upstream/Downstream)	*1,618 gpm/1,775 gpm*
Total Project Cost	*$829,000*
Prior Spending	*$0*
Costs in this Plan Period	*$829,000*

Pro Rata Area	*Accokeek Creek*

A-34: Construct 21-inch gravity main along Rocky Run from vicinity of Rockdale Road to vicinity of Boulder Drive (3,913 feet)

This project includes design and construction of a 21-inch gravity main along Rocky Run from vicinity of Rockdale Road to vicinity of Boulder Drive (3,913 feet). The purpose of the project is to serve future customers in the vicinity of Rocky Run and convey flows from the Upper Accokeek PS and Route 630 PS. The timing for construction of this project is dependent on the timing of flows in the Rocky Run area which is anticipated to be in the near-term (prior to 2010).

Priority	*2 – Near-term*
Design	*2007*
Construct	*2008*
Design Flow (Upstream/Downstream)	*3,653 gpm/3,715 gpm*
Total Project Cost	*$964,000*
Prior Spending	*$0*
Costs in this Plan Period	*$964,000*
Pro Rata Area	*Accokeek Creek & Austin Run*

A-35: Construct 24-inch gravity main along Rocky Run from vicinity of Boulder Drive to interceptor at Austin Ridge Drive (785 feet)

This project includes design and construction of a 24-inch gravity main along Rocky Run from vicinity of Boulder Drive to interceptor at Austin Ridge Drive (785 feet). The purpose of the project is to serve future customers in the vicinity of Rocky Run and convey flows from the Upper Accokeek PS and Route 630 PS. The timing for construction of this project is dependent on the timing of flows in the Rocky Run area which is anticipated to be in the near-term (prior to 2010).

Priority	*2 – Near-term*
Design	*2007*
Construct	*2008*
Design Flow (Upstream/Downstream)	*3,721 gpm/3,724 gpm*
Total Project Cost	*$216,000*
Prior Spending	*$0*
Costs in this Plan Period	*$216,000*
Pro Rata Area	*Accokeek Creek & Austin Run*

A-36: Construct 8-inch gravity main along Rocky Run from 21-inch main near Rockdale Road to vicinity of Embrey Mill Road (5,741 feet)

This project includes design and construction of an 8-inch gravity main along Rocky Run from 21-inch main near Rockdale Road to vicinity of Embrey Mill Road (5,741 feet). The purpose of the project is to serve future customers in the vicinity of Rocky Run. The timing for construction of this project is dependent on the timing of flows in the Rocky Run area which is anticipated to be in the near-term (prior to 2010).

Priority	*2 – Near-term*
Design	*2008*
Construct	*2009*
Design Flow (Upstream/Downstream)	*75 gpm/353 gpm*
Total Project Cost	*$781,000*
Prior Spending	*$0*

Costs in this Plan Period	*$781,000*
Pro Rata Area	*None*

A-37: Construct 8-inch gravity main from interceptor along Austin Run near Winding Creek Road and Marshall Road to Heritage Oaks II PS (2,472 feet)

This project includes design and construction of an 8-inch gravity main from interceptor along Austin Run near Winding Creek Road and Marshall Road to Heritage Oaks II PS (2,472 feet). The purpose of the project is to eliminate the Heritage Oaks II PS. The timing for construction of this project is dependent on the available capacity of the Heritage Oaks II PS which is anticipated to be fully utilized by roughly 2018.

Priority	*1 - Operations*
Design	*2017*
Construct	*2018*
Design Flow (Upstream/Downstream)	*134 gpm/148 gpm*
Total Project Cost	*$336,000*
Prior Spending	*$0*
Costs in this Plan Period	*$336,000*
Pro Rata Area	*None*

A-38: Replace 10-inch and 12-inch with 18-inch gravity main along Garrisonville Road and unnamed tributary to Whitsons Run from Onville Road to interceptor along Whitsons Run (5,050 feet)

This project includes replacement of the existing 10-inch and 12-inch with 18-inch gravity main along Garrisonville Road and unnamed tributary to Whitsons Run from Onville Road to interceptor along Whitsons Run (5,050 feet). The purpose of the project is to increase the conveyance capacity of the existing 10-inch and 12-inch gravity mains to handle flows from Quantico Marine Corps Base. The existing 10-inch and 12-inch mains are roughly 40% to 60% full under near-term conditions (assuming 1.5 mgd flow from Quantico Marine Corps Base) and exceeds full flow under buildout conditions. The timing for construction of this project is dependent on the timing of flows from Quantico Marine Corps Base.

Priority	*3 - Buildout*
Design	*2012*
Construct	*2013*
Design Flow (Upstream/Downstream)	*3,645 gpm/3,930 gpm*
Total Project Cost	*$1,669,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,669,000*
Pro Rata Area	*Austin Run*

A-39: Replace 18-inch with 24-inch gravity main along Whitsons Run from vicinity of Highpointe Boulevard to interceptor along Austin Run (6,890 feet)

This project includes replacement of the existing 18-inch with 24-inch gravity main along Whitsons Run from vicinity of Highpointe Boulevard to interceptor along Austin Run (6,890 feet). The purpose of the project is to increase the conveyance capacity of the existing 18-inch gravity mains to handle flows from Quantico Marine Corps Base. . The existing 18-inch main is roughly 50% to 60% full under near-term conditions (assuming 1.5 mgd flow from Quantico Marine Corps Base) and exceeds full flow under buildout conditions. The timing for construction of this project is dependent on the timing of flows from Quantico Marine Corps Base.

Priority	*3 - Buildout*
Design	*2012*
Construct	*2013*
Design Flow (Upstream/Downstream)	*5,178 gpm/6,138 gpm*
Total Project Cost	*$2,846,000*
Prior Spending	*$0*
Costs in this Plan Period	*$2,846,000*
Pro Rata Area	*Austin Run*

A-40: Replace 8-inch with 12-inch gravity main along Aquia Drive from Delaware Drive to Vessel Drive (1,929 feet)

This project includes replacement of the existing 8-inch with 12-inch gravity main along Aquia Drive from Delaware Drive to Vessel Drive (1,929 feet). The purpose of the project is to increase the conveyance capacity of the existing 8-inch gravity main. Prior to replacing the existing gravity main, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing gravity main.

Priority	*7 – Flow monitoring*
Design	*2014*
Construct	*2014*
Design Flow (Upstream/Downstream)	*326 gpm/460 gpm*
Total Project Cost	*$375,000*
Prior Spending	*$0*
Costs in this Plan Period	*$375,000*
Pro Rata Area	*Aquia Creek*

A-42: Replace 8-inch with 18-inch gravity main along Jefferson Davis Highway from Aquia Creek to Potomac Hills Drive (5,298 feet)

This project includes replacement of the existing 8-inch with 18-inch gravity main along Jefferson Davis Highway from Aquia Creek to Potomac Hills Drive (5,298 feet). The purpose of the project is to significantly increase the capacity of the interceptor serving the northern portion of the Jefferson Davis Highway corridor. The County has appropriated funds for this project.

Priority	*5 – Prior Appropriation*
Design	*2005*
Construct	*2005*
Design Flow (Upstream/Downstream)	*747 gpm/1,492 gpm*
Total Project Cost	*$1,390,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,390,000*
Pro Rata Area	*Aquia Creek*

A-44: Replace 8-inch with 18-inch gravity main along Dewey Drive from Aquia Drive to Aquia Drive (2,259 feet)

This project includes replacement of the existing 8-inch with 18-inch gravity main along Dewey Drive from Aquia Drive to Aquia Drive (2,259 feet). The purpose of this project is to increase the capacity of the existing 8-inch gravity main. Prior to replacing the existing gravity main, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing gravity main.

Priority	*7 – Flow Monitoring*
Design	*2014*
Construct	*2014*
Design Flow (Upstream/Downstream)	*655 gpm/677 gpm*
Total Project Cost	*$593,000*
Prior Spending	*$0*
Costs in this Plan Period	*$593,000*
Pro Rata Area	*Aquia Creek*

A-45: Replace 8-inch with 12-inch gravity main along Aquia Drive from Schooner Drive to vicinity of Anchor Cove (1,376 feet)

This project includes replacement of the existing 8-inch with 12-inch gravity main along Aquia Drive from Schooner Drive to vicinity of Anchor Cove (1,376 feet). The purpose of this project is to increase the capacity of the existing 12-inch gravity main. Prior to replacing the existing gravity main, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing gravity main.

Priority	*7 – Flow Monitoring*
Design	*2014*
Construct	*2014*
Design Flow (Upstream/Downstream)	*615 gpm/633 gpm*
Total Project Cost	*$267,000*
Prior Spending	*$0*
Costs in this Plan Period	*$267,000*
Pro Rata Area	*Aquia Creek*

A-47: Replace 8-inch with 15-inch gravity main near Voyage Drive (686 feet)

This project includes replacement of the existing 8-inch with 15-inch gravity main near Voyage Drive (686 feet). The purpose of this project is to increase the capacity of the existing 8-inch gravity main. Prior to replacing the existing gravity main, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing gravity main.

Priority	*7 – Flow Monitoring*
Design	*2014*
Construct	*2014*
Design Flow (Upstream/Downstream)	*441 gpm/444 gpm*
Total Project Cost	*$157,000*
Prior Spending	*$0*
Costs in this Plan Period	*$157,000*
Pro Rata Area	*Aquia Creek*

A-48: Construct 8-inch gravity main to serve area near Sheron Lane to PS along Aquia Creek (4,000 feet)

This project includes design and construction of an 8-inch gravity main to serve area near Sheron Lane to PS along Aquia Creek (4,000 feet). The purpose of the project is to serve future customers in this area.

Priority	*3 - Buildout*
Design	*2022*
Construct	*2023*

Design Flow (Upstream/Downstream)	*27 gpm/43 gpm*
Total Project Cost	*$544,000*
Prior Spending	*$0*
Costs in this Plan Period	*$544,000*
Pro Rata Area	*Aquia Creek*

A-49: Construct 8-inch gravity main to serve area west of Country Ridge to PS along Aquia Creek (4,000 feet)

This project includes design and construction of an 8-inch gravity main to serve the area west of Country Ridge to PS along Aquia Creek (4,000 feet). The purpose of this project is to serve future customers in this area.

Priority	*3 – Buildout*
Design	*2022*
Construct	*2023*
Design Flow (Upstream/Downstream)	*11 gpm/26 gpm*
Total Project Cost	*$544,000*
Prior Spending	*$0*
Costs in this Plan Period	*$544,000*
Pro Rata Area	*Aquia Creek*

7.6.2. Aquia WWTP - Force Mains

A-100: Construct 16-inch force main along Cedar Lane from Upper Accokeek PS to Rocky Run Interceptor (6,526 feet)

This project includes design and construction of a 16-inch force main along Cedar Lane from Upper Accokeek PS to Rocky Run Interceptor (6,526 feet). The purpose of the project is to convey flows from the Upper Accokeek PS to the Rocky Run Interceptor. The timing for construction of this project is dependent on the timing of improvements to the Upper Accokeek PS.

Priority	*2 – Near-term*
Design	*2009*
Construct	*2010*
Design Flow	*2,645 gpm*
Total Project Cost	*$740,000*
Prior Spending	*$0*
Costs in this Plan Period	*$740,000*
Pro Rata Area	*Accokeek Creek*

A-102: Construct 6-inch force main from North Stafford PS to Upper Accokeek PS (1,729 feet)

This project includes design and construction of a 6-inch force main from North Stafford PS to Upper Accokeek PS (1,729 feet). Currently, the Upper Accokeek PS conveys flow to the North Stafford PS for pumping through a force main along Jefferson Davis Highway. Following construction of the Rocky Run Interceptor on the western side of I-95, a short segment of 6-inch force main near the North Stafford PS will be constructed and the flow from the North Stafford PS will then be pumped to the Upper Accokeek PS which will be subsequently pumped to the Rocky Run Interceptor. The purpose of the project is to convey flow from the North Stafford PS to the existing 6-inch force main connected to the Upper Accokeek PS. The timing for construction of this project is dependent on the timing for construction of the Rocky Run Interceptor and the proposed 16-inch force main from the Upper Accokeek PS. It is anticipated that the Rocky Run Interceptor will be constructed in the near-term (prior to 2010).

Priority	*2 – Near-term*
Design	*2009*
Construct	*2010*
Design Flow	*72 gpm*
Total Project Cost	*$101,000*
Prior Spending	*$0*
Costs in this Plan Period	*$101,000*
Pro Rata Area	*Accokeek Creek*

A-103: Construct 12-inch force main along Jefferson Davis Highway from Lower Accokeek PS (12,248 feet)

This project includes design and construction of a 12-inch force main along Jefferson Davis Highway from Lower Accokeek PS (12,248 feet). The purpose of the project is to convey flows from the Lower Accokeek PS which will serve future customers in the Accokeek basin east of I-95. The timing for construction of this project is dependent on the timing of water demands in the area which is anticipated to be after 2015.

Priority	*3 - Buildout*
Design	*2021*
Construct	*2022*

Design Flow	*1,775 gpm*
Total Project Cost	*$1,052,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,052,000*
Pro Rata Area	*Accokeek Creek*

A-104: Replace 16-inch with dual 24-inch force mains along Austin Run from Austin Run PS to Aquia WWTP (927 feet for each force main)

This project includes replacement of the existing 16-inch with dual 24-inch force mains along Austin Run from Austin Run PS to Aquia WWTP (1,854 feet). The purpose of the project is to increase the conveyance capacity immediately upstream of the Austin Run PS. The timing for construction of this project is based on maintaining an acceptable velocity in the existing 16-inch force main. It is anticipated that the near-term flow of roughly 15 mgd will significantly exceed the conveyance capacity of the existing 16-inch force main. Alternatively, DOU has been considering the possibility of constructing a large-diameter gravity sewer downstream of the Austin Run PS to convey flow to the Aquia WWTP.

Priority	*2 – Near-term*
Design	*2008*
Construct	*2009*
Design Flow	*21,002 gpm (11,000 gpm each)*
Total Project Cost	*$276,000*
Prior Spending	*$0*
Costs in this Plan Period	*$276,000*
Pro Rata Area	*Accokeek Creek & Austin Run*

A-106: Construct 4-inch force main along Courthouse Road from Route 630 PS to Cedar Lane (2,833 feet)

This project includes design and construction of a 4-inch force main along Courthouse Road from Route 630 PS to Cedar Lane (2,833 feet). Currently, the Route 630 PS conveys flow under I-95 to the Oaks of Stafford PS and Courthouse PS. To reduce repumping and potentially eliminate the need for improvements to the downstream pumping stations, it is recommended that the 4-inch force main be constructed along Courthouse Road to the proposed Rocky Run Interceptor. The timing for construction of this project is dependent on the timing for construction of the Rocky Run Interceptor. It is anticipated that this project will not be needed in the near-term (prior to 2010).

Priority	*3 - Buildout*
Design	*2018*
Construct	*2019*
Design Flow	*217 gpm*
Total Project Cost	*$138,000*
Prior Spending	*$0*
Costs in this Plan Period	*$138,000*

A-112: Construct 6-inch force main from Sheron Lane PS near Aquia Creek (7,984 feet)

This project includes design and construction of a 6-inch force main from Sheron Lane PS near Aquia Creek (7,984 feet). The purpose of the project is to serve future customers in the area near Sheron Lane. The timing for construction of this project is dependent on the timing of flows in this area and construction of the Sheron Lane PS (A-231).

Priority	*3 - Buildout*

Design	*2022*
Construct	*2023*
Design Flow	*69 gpm*
Total Project Cost	*$466,000*
Prior Spending	*$0*
Costs in this Plan Period	*$466,000*
Pro Rata Area	

A-114: Replace 8-inch and 10-inch force mains from Aquia Creek PS at Crucifix to existing 14-inch force main near Aquia Drive (2,600 feet)

This project includes replacement of the 8-inch and 10-inch force mains from Aquia Creek PS at Crucifix to existing 14-inch force main near Aquia Drive (2,600 feet). The purpose of the project is to alleviate capacity concerns in the 8-inch and 10-inch force mains. DOU is currently replacing the 8-inch and 10-inch force mains.

Priority	*2 – Near-term*
Design	*2006*
Construct	*2007*
Design Flow	*3,250 gpm*
Total Project Cost	*$305,000*
Prior Spending	*$0*
Costs in this Plan Period	*$305,000*
Pro Rata Area	*Aquia Creek*

A-115: Replace 14-inch and 12-inch force mains from Aquia at Bridge PS to existing 18-inch force main near Starboard Cove Lane (6,976 feet)

This project includes replacement of the 14-inch and 12-inch force mains from Aquia at Bridge PS to the existing 18-inch force main near Starboard Cove Lane (6,976 feet). The purpose of the project is to increase the capacity of the force main for the buildout condition. The timing for construction of this project is dependent on the timing of flows in this area.

Priority	*3 - Buildout*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Design Flow	*1,465 gpm*
Total Project Cost	*$819,000*
Prior Spending	*$0*
Costs in this Plan Period	*$819,000*
Pro Rata Area	*Aquia Creek*

7.6.3. *Aquia WWTP – Pumping Stations*

A-202: Expand Garrisonville PS to 5.6 mgd

This project includes expansion of the Garrisonville PS from 3.5 mgd to 5.6 mgd at buildout. For the buildout condition, it was concluded from modeling that the capacity (assuming full flow) of the interceptor downstream of the force main serving the Garrisonville PS could accept a maximum of approximately 700 gpm (1 mgd) of flow from the area west of the Garrisonville PS (i.e., vicinity of Lake Arrowhead). Flows in excess of roughly 1 mgd would require upsizing the existing interceptors along Austin Run and Blossom Wood Court at a cost of roughly $1.6 million. Under buildout conditions, the existing interceptor downstream of the force main serving Garrisonville PS flows between 55% and 80% full. It is anticipated that flows from the area west of the Garrisonville PS would not be conveyed to the Garrisonville PS until 2025 or beyond. Note that although the flows from the area west of the Garrisonville PS (i.e., vicinity of Lake Arrowhead) were included in the Garrisonville PS and downstream facilities, the facilities and costs for the facilities needed to transport flow from the Lake Arrowhead area to the Garrisonville PS have not been included.

Priority	*3 – Buildout*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Design Flow	*3,892 gpm*
Total Project Cost	*$722,000*
Prior Spending	*$0*
Costs in this Plan Period	*$722,000*
Pro Rata Area	*Austin Run*

A-203: Expand Heritage Oaks PS No. 1 to 0.24 mgd

This project includes expansion of the Heritage Oaks PS No. 1 from 0.15 mgd to 0.24 mgd prior to buildout (i.e., prior to taking Heritage Oaks PS No. 2 out of service). This pumping station serves an area that is currently developed and served with public sewer. Flow projections indicate that this pumping station may have insufficient capacity to meet the near-term flows. In 2017-2018, it is recommended that the Heritage Oaks PS No.2, which conveys flow to the Heritage Oaks PS No. 1, be abandoned and a gravity sewer constructed to carry flow to the interceptor along Austin Run. Offloading the flow from Heritage Oaks PS No. 2 may defer or eliminate the need for expansion of the Heritage Oaks PS No. 1. Prior to expanding the existing Heritage Oaks PS No. 1, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing pumping station.

Priority	*7 – Flow Monitoring*
Design	*2014*
Construct	*2014*
Total Project Cost	*$54,000*
Prior Spending	*$0*
Costs in this Plan Period	*$54,000*

A-205: Expand Upper Accokeek PS to 3.8 mgd

This project includes expansion of the Upper Accokeek PS from 0.25 mgd to 3.8 mgd. Currently, the Upper Accokeek PS conveys flow to the North Stafford PS for pumping through a force main along Jefferson Davis Highway. Following construction of the Rocky Run Interceptor on the western side of I-95, a 16-inch force main will be constructed to convey flows from the Upper Accokeek PS to the Rocky Run Interceptor. The timing for construction of this project is dependent on the timing for construction of the Rocky Run Interceptor and the proposed 16-inch force main from the Upper Accokeek PS. Flow projections and modeling indicate that the pumping station may have insufficient capacity to meet near-term flows. It is anticipated that the Rocky Run Interceptor and the expansion of the Upper Accokeek PS will be constructed in the near-term (prior to 2010).

Priority	*2 – Near-term*
Design	*2009*
Construct	*2010*
Design Flow	*2,645 gpm*
Total Project Cost	*$1,245,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,245,000*
Pro Rata Area	*Accokeek Creek*

A-207: Construct Lower Accokeek PS at 2.6 mgd

This project includes design and construction of the Lower Accokeek PS at 2.6 mgd. The purpose of the project is to serve future customers in the vicinity of the Lower Accokeek PS and convey flows from the Wyche Road PS and the Rowser PS which will be abandoned. It is anticipated that development generally east of Jefferson Davis Highway in the vicinity of the Lower Accokeek PS will not occur until after 2015.

Priority	*3 – Buildout*
Design	*2021*
Construct	*2022*
Design Flow	*1,775 gpm*
Total Project Cost	*$895,000*
Prior Spending	*$0*
Costs in this Plan Period	*$895,000*
Pro Rata Area	*Accokeek Creek*

A-208: Expand Oaks of Stafford PS to 0.38 mgd

This project includes expansion of the Oaks of Stafford PS from 0.26 mgd to 0.38 mgd. This pumping station serves an area that is partially developed and served with public sewer. Flow projections and modeling indicate that the pumping station should have sufficient capacity to meet near-term flows. Prior to expanding the existing pumping station, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing pumping station.

Priority	*7 – Flow Monitoring*
Design	*2014*
Construct	*2014*
Design Flow	*266 gpm*
Total Project Cost	*$214,000*
Prior Spending	*$0*
Costs in this Plan Period	*$214,000*
Pro Rata Area	*Austin Run*

A-209: Expand Route 630 PS to 0.31 mgd

This project includes expansion of the Route 630 PS from 0.14 mgd to 0.31 mgd. Flow projections and modeling indicate that the pumping station should have sufficient capacity to meet near-term flows. The timing for expansion of the pumping station should be concurrent with construction of the proposed force main (A-106) to the Rocky Run Interceptor.

Priority	*3 – Buildout*
Design	*2018*
Construct	*2019*
Design Flow	*217 gpm*
Total Project Cost	*$101,000*
Prior Spending	*$0*
Costs in this Plan Period	*$101,000*
Pro Rata Area	*Austin Run*

A-210: Expand Austin Run PS to 30 mgd

This project includes expansion of the Austin Run PS from 5.8 mgd to 30 mgd. Flow projections and modeling indicate that the pumping station will have insufficient capacity to meet near-term flows (roughly 14.7 mgd). It is recommended that the pumping station be expanded from 5.8 mgd to approximately 20 mgd in 2006-2007 and from 20 mgd to 30 mgd beyond 2025. In lieu of expanding the Austin Run PS, DOU has been considering the possibility of constructing a large-diameter gravity sewer downstream of the Austin Run PS to convey flow to the Aquia WWTP.

Priority	*1 – Operations*
Design	*2006/Beyond 2025*
Construct	*2007/Beyond 2025*
Design Flow	*21,002 gpm*
Total Project Cost	*$4,891,000*
Prior Spending	*$0*
Costs in this Plan Period	*$4,891,000*
Pro Rata Area	*Austin Run*

A-212: Expand Aquia Creek PS to 4.68 mgd

This project includes expansion of the Aquia Creek PS from 2.88 mgd to 4.68 mgd. Flow projections and modeling indicate that the pumping station will have insufficient capacity to meet near-term flows. Prior to expanding the existing pumping station, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing pumping station.

Priority	*2 – Near-term*
Design	*2008*
Construct	*2009*
Design Flow	*3,250 gpm*
Total Project Cost	*$630,000*
Prior Spending	*$0*
Costs in this Plan Period	*$630,000*
Pro Rata Area	*Aquia Creek*

A-213: Construct Hildrups PS at 0.64 mgd

This project includes design and construction of the Hildrups PS at 0.64 mgd. The purpose of the project is to serve future customers in the northern portion of the Jefferson Davis Highway corridor. It is anticipated that this pumping station will be needed in the near-term (prior to 2010).

Priority	*2 – Near-term*
Design	*2006*
Construct	*2007*
Design Flow	*444 gpm*
Total Project Cost	*$384,000*
Prior Spending	*$0*
Costs in this Plan Period	*$384,000*
Pro Rata Area	*Aquia Creek*

A-214: Expand Stonebridge PS to 0.45 mgd

This project includes expansion of the Stonebridge PS from 0.32 mgd to 0.45 mgd. This pumping station serves an area that is partially developed and is served by public sewer. Flow projections and modeling indicate that the pumping station will have sufficient capacity to meet near-term flows.

Priority	*3 – Buildout*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Design Flow	*314 gpm*
Total Project Cost	*$81,000*
Prior Spending	*$0*
Costs in this Plan Period	*$81,000*
Pro Rata Area	*Accokeek Creek*

A-216: Expand Aquia at Bridge PS to 2.11 mgd

This project includes expansion of the Aquia at Bridge from 1.89 mgd to 2.11 mgd. This pumping station serves an area that is partially developed and is served by public sewer. Flow projections and modeling indicate that the pumping station will have sufficient capacity to nearly meet the buildout flows.

Priority	*3 – Buildout*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Design Flow	*1,465 gpm*
Total Project Cost	*$134,000*
Prior Spending	*$0*
Costs in this Plan Period	*$134,000*
Pro Rata Area	*Accokeek Creek*

A-217: Expand Courthouse PS to 1.52 mgd

This project includes expansion of the Courthouse PS from 1.3 mgd to 1.52 mgd. Flow projections and modeling indicate that the pumping station will have insufficient capacity to meet near-term flows. However, connecting the existing force main to the proposed 18-inch interceptor (A-30) in the vicinity of Somerset Lane should improve pumping capabilities. In addition, rerouting flows from the Route 630 PS to the Rocky Run Interceptor will provide additional capacity at the Courthouse PS. These modifications may provide sufficient capacity to meet buildout flows. Prior to expanding the existing pumping station, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing pumping station.

Priority	*7 – Flow Monitoring*
Design	*2014*
Construct	*2014*

Design Flow	*1,058 gpm*
Total Project Cost	*$136,000*
Prior Spending	*$0*
Costs in this Plan Period	*$136,000*
Pro Rata Area	*Austin Run*

<u>A-218: Expand Anchor Cove PS to 0.88 mgd</u>

This project includes expansion of the Anchor Cove PS from 0.59 mgd to 0.88 mgd. Flow projections and modeling indicate that the pumping station will have insufficient capacity to meet near-term flows. This pumping station serves an area that is partially developed and is served by public sewer. Prior to expanding the existing pumping station, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing pumping station.

Priority	*7 – Flow Monitoring*
Design	*2014*
Construct	*2014*
Design Flow	*610 gpm*
Total Project Cost	*$173,000*
Prior Spending	*$0*
Costs in this Plan Period	*$173,000*
Pro Rata Area	*Aquia Creek*

<u>A-219: Expand Foresail Cove PS to 0.65 mgd</u>

This project includes expansion of the Foresail Cove PS from 0.59 mgd to 0.65 mgd. Flow projections and modeling indicate that the pumping station will have insufficient capacity to meet near-term flows. This pumping station serves an area that is partially developed and is served by public sewer. Prior to expanding the existing pumping station, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing pumping station.

Priority	*7 – Flow Monitoring*
Design	*2014*
Construct	*2014*
Design Flow	*450 gpm*
Total Project Cost	*$35,000*
Prior Spending	*$0*
Costs in this Plan Period	*$35,000*
Pro Rata Area	*Aquia Creek*

<u>A-224: Expand Camp Barrett PS to 5.25 mgd</u>

This project includes expansion of the Camp Barrett PS to 5.25 mgd. The Camp Barrett PS serves the Quantico Marine Corps Base and the timing for expansion of the existing pumping station is dependent on the flows generated by the Quantico Marine Corps Base. For planning purposes, it is assumed that the expansion would not be needed in the near-term (prior to 2010).

Priority	*3 – Buildout*
Design	*2012*
Construct	*2013*
Design Flow	*3,644 gpm*

Total Project Cost	*$1,050,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,050,000*
Pro Rata Area	*None*

A-231: Construct Sheron Lane PS at 0.10 mgd

This project includes design and construction of the Sheron Lane PS at 0.10 mgd. The purpose of the project is to serve future customers in the area near Sheron Lane. The timing for construction of this project is dependent on the timing of flows in this area which are anticipated to occur after 2015.

Priority	*3 – Buildout*
Design	*2022*
Construct	*2023*
Design Flow	*69 gpm*
Total Project Cost	*$60,000*
Prior Spending	*$0*
Costs in this Plan Period	*$60,000*

7.7. Little Falls Run Wastewater Treatment Facility Service Area

7.7.1. Little Falls Run WWTP - Gravity Mains

LFR-1: Replace 18-inch with 36-inch gravity main along Falls Run from Warrenton Road near Jefferson Davis Highway to vicinity of Kelley Road and Truslow Road (3,427 feet)

This project includes replacement of the existing 18-inch with 36-inch gravity main along Falls Run from Warrenton Road near Jefferson Davis Highway to vicinity of Kelley Road and Truslow Road (3,427 feet). The purpose of the project is to significantly increase the conveyance capacity of interceptor along Falls Run to satisfy future needs. The timing for this project is dependent on the timing for development of the area along England Run, Westlake, and the area along Potomac Creek west of Abel Lake. Of these three developments, it is anticipated that only the area along England Run will be developed in the near-term (prior to 2010). The existing 18-inch main is roughly 60% full under near-term (2010) flow conditions and exceeds full flow under buildout conditions.

Priority	*3 - Buildout*
Design	*2014*
Construct	*2015*
Design Flow (Upstream/Downstream)	*9,399 gpm/9,729 gpm*
Total Project Cost	*$1,393,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,393,000*
Pro Rata Areas	*Falls Run, Horse Pen-RPR*

LFR-2: Replace 18-inch with 30-inch gravity main along Falls Run from 36-inch near Kelley Road and Truslow Road to vicinity of Stanstead Road (8,494 feet)

This project includes replacement of the existing 18-inch with 30-inch gravity main along Falls Run from 36-inch near Kelley Road and Truslow Road to vicinity of Stanstead Road (8,494 feet). The purpose of the project is to significantly increase the conveyance capacity of interceptor along Falls Run to satisfy future needs. The timing for this project is dependent on the timing for development of the area along England Run, Westlake, and the area along Potomac Creek west of Abel Lake. Of these three developments, it is anticipated that only the area along England Run will be developed in the near-term (prior to 2010). The existing 18-inch main is roughly 55% full under near-term (2010) flow conditions and exceeds full flow under buildout conditions.

Priority	*3 - Buildout*
Design	*2014*
Construct	*2015*
Design Flow (Upstream/Downstream)	*6,068 gpm/9,356 gpm*
Total Project Cost	*$3,608,000*
Prior Spending	*$0*
Costs in this Plan Period	*$3,608,000*
Pro Rata Area	*Falls Run, Horse Pen-RPR*

LFR-3: Replace 15-inch and 12-inch with 24-inch gravity main along Falls Run from 30-inch in vicinity of Stanstead Road to Pennsbury Court (13,090 feet)

This project includes replacement of the existing 15-inch and 12-inch with 24-inch gravity main along Falls Run from 30-inch in vicinity of Stanstead Road to Pennsbury Court (13,090 feet). The purpose of the project is to significantly increase the conveyance capacity of interceptor along Falls Run to satisfy future needs. The timing for this project is dependent on the timing for development of Westlake and the area along Potomac Creek west of Abel Lake. It is anticipated that these areas will not be developed in the near-term (prior to 2010). The existing 15-inch and 12-inch mains are generally between 30% and 50% full under near-term (2010) flow conditions and exceed full flow under buildout conditions.

Priority	*3 - Buildout*
Design	*2014*
Construct	*2015*
Design Flow (Upstream/Downstream)	*3,080 gpm/5,619 gpm*
Total Project Cost	*$5,407,000*
Prior Spending	*$0*
Costs in this Plan Period	*$5,407,000*
Pro Rata Area	*Falls Run, Horse Pen-RPR*

LFR-4: Replace 12-inch with 24-inch gravity main along I-95 from force main serving England Run PS to Falls Run interceptor (2,615 feet)

This project includes replacement of the existing 12-inch with 24-inch gravity main along I-95 from force main serving England Run PS to Falls Run interceptor (2,615 feet). The timing for this project should be concurrent with the expansion of the England Run PS (LFR-204) . It is anticipated that development of the area along England Run and construction of the sewer facilities will occur between 2010 and 2020. .

Priority	*2 – Near-term*
Design	*2016*
Construct	*2017*
Design Flow (Upstream/Downstream)	*2,317 gpm/2,399 gpm*
Total Project Cost	*$1,080,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,080,000*
Pro Rata Area	*Falls Run*

LFR-12: Replace 15-inch with 21-inch gravity main along Potomac Creek from vicinity of I-95 to Potomac Creek PS (6,000 feet)

This project includes replacement of the existing 15-inch with 21-inch gravity main along Potomac Creek from vicinity of I-95 to Potomac Creek PS (6,000 feet). The existing 15-inch main is roughly 25% to 30% full under near-term (2010) flow conditions and exceeds full flow under buildout conditions.

Priority	*3 - Buildout*
Design	*2019*
Construct	*2020*
Design Flow (Upstream/Downstream)	*2,004 gpm/3,015 gpm*
Total Project Cost	*$1,798,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,798,000*
Pro Rata Area	*Potomac Creek*

LFR-14: Replace 18-inch and 24-inch with 27-inch gravity main along Claiborne Run from vicinity of White Oak Road to Morton Road (12,117 feet)

This project includes replacement of the existing 18-inch and 24-inch with 27-inch gravity main along Claiborne Run from vicinity of White Oak Road to Morton Road (12,117 feet). The existing 18-inch and 24-inch mains are roughly 25% to 30% full under near-term (2010) flow conditions and exceed full flow under buildout conditions. The timing for this project will be dependent on the timing of flows from the

Potomac Creek PS. It is anticipated that the capacity of the existing mains will not be exceeded until after 2025. However, the County will be replacing the segment of interceptor along Claiborne Run downstream of this segment due to poor structural condition (LFR-42). The County may decide to replace the 18-inch and 24-inch mains earlier in the planning period if these mains are also found to be in poor condition.

Priority	*3 - Buildout*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Design Flow (Upstream/Downstream)	*5,278 gpm/6,816 gpm*
Total Project Cost	*$4,260,000*
Prior Spending	*$0*
Costs in this Plan Period	*$4,260,000*
Pro Rata Area	*Claiborne Run, Potomac Creek*

LFR-15: Replace 18-inch, 15-inch and 12-inch with 24-inch gravity main along Claiborne Run from Morton Road to Kings Hill Road (4,000 feet)

This project includes replacement of the existing 18-inch, 15-inch and 12-inch with 24-inch gravity main along Claiborne Run from Morton Road to Kings Hill Road (4,000 feet). The existing 18-inch, 15-inch and 12-inch mains are roughly 40% to 50% full under near-term (2010) flow conditions and exceed full flow under buildout conditions. The timing for this project will be dependent on the timing of flows from the Potomac Creek PS. It is anticipated that the capacity of the existing mains will not be exceeded until after 2025. However, the County will be replacing the segment of interceptor along Claiborne Run upstream of the Claiborne Run PS due to poor structural condition (LFR-42). The County may decide to replace the 18-inch, 15-inch and 12-inch mains earlier in the planning period if these mains are also found to be in poor condition.

Priority	*3 - Buildout*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Design Flow (Upstream/Downstream)	*3,742 gpm/4,144 gpm*
Total Project Cost	*$1,382,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,382,000*
Pro Rata Area	*Falls Run, Potomac Creek*

LFR-19: Construct 10-inch gravity main along unnamed tributary to Claiborne Run from Blythedale PS to Claiborne Run PS near Cool Springs Road (1,490 feet)

This project includes design and construction of a 10-inch gravity main along unnamed tributary to Claiborne Run from Blythedale PS to Claiborne Run PS near Cool Springs Road (1,490 feet). The purpose of the project is to eliminate the Blythedale PS; thereby eliminating the need for pumping and providing additional conveyance capacity in the gravity piping downstream of the existing force main serving the Blythedale PS. The County has appropriated funds for this project.

Priority	*5 – Prior Appropriation*
Design	*2005*
Construct	*2005*
Design Flow (Upstream/Downstream)	*448 gpm/462 gpm*
Total Project Cost	*$361,000*
Prior Spending	*$0*

Costs in this Plan Period	*$361,000*
Pro Rata Area	*Claiborne Run*

LFR-22: Construct 10-inch gravity main from force main serving Upper Potomac Creek PS No. 1 to Falls Run interceptor near Berea Church Road (3,500 feet)

This project includes design and construction of a 10-inch gravity main from force main serving Upper Potomac Creek PS No. 1 to Falls Run interceptor near Berea Church Road (3,500 feet). The purpose of the project is to serve future customers in this area. Due to the significant improvements needed for the interceptor along Falls Run which serves this area, it is recommended that the timing for construction of sewer facilities in Westlake and the area along Potomac Creek west of Abel Lake be deferred until development in the area warrants replacement of the Falls Run Interceptor. It is anticipated that development of the area along Potomac Creek west of Abel Lake and construction of the associated sewer facilities will occur after 2015.

Priority	*3 - Buildout*
Design	*2022*
Construct	*2023*
Design Flow (Upstream/Downstream)	*263 gpm/302 gpm*
Total Project Cost	*$527,000*
Prior Spending	*$0*
Costs in this Plan Period	*$527,000*
Pro Rata Area	*None*

LFR-23: Construct 10-inch gravity main along unnamed tributary to England Run in vicinity of Riverside Parkway and Sanford Drive (3,000 feet)

This project includes design and construction of a 10-inch gravity main along unnamed tributary to England Run in vicinity of Riverside Parkway and Sanford Drive (3,000 feet). The purpose of the project is to serve development south of Warrenton Road and eliminate the Heritage CC PS and the Days Inn PS. The timing for construction of this project is dependent on development in the area and the need to expand or maintain the existing pumping stations.

Priority	*3 - Buildout*
Design	*2020*
Construct	*2021*
Design Flow (Upstream/Downstream)	*232 gpm/314 gpm*
Total Project Cost	*$452,000*
Prior Spending	*$0*
Costs in this Plan Period	*$452,000*
Pro Rata Area	*Falls Run*

LFR-24: Construct 12-inch gravity main along Horsepen Run from Westlake PS in vicinity of Cedar Grove Road (3,615 feet)

This project includes design and construction of a 12-inch gravity main along Horsepen Run from the Westlake PS in the vicinity of Cedar Grove Road (3,615 feet). The purpose of the project is to serve future customers in this area. Due to the significant improvements needed for the interceptor along Falls Run which serves this area, it is recommended that the timing for construction of sewer facilities in Westlake and the area along Potomac Creek west of Abel Lake be deferred until development in the area warrants replacement of the Falls Run Interceptor. It is anticipated that development of Westlake and construction of the associated sewer facilities will occur after 2015.

Priority	*3 - Buildout*
Design	*2017*
Construct	*2018*
Design Flow (Upstream/Downstream)	*525 gpm/756 gpm*
Total Project Cost	*$597,000*
Prior Spending	*$0*
Costs in this Plan Period	*$597,000*
Pro Rata Area	*Horse Pen-RPR*

LFR-25: Construct 8-inch gravity main along unnamed tributary to Horsepen Run from Westlake PS to vicinity of Clark Patton Road (3,842 feet)

This project includes design and construction of an 8-inch gravity main along unnamed tributary to Horsepen Run from the Westlake PS to the vicinity of Clark Patton Road (3,842 feet). The purpose of the project is to serve future customers in this area. Due to the significant improvements needed for the interceptor along Falls Run which serves this area, it is recommended that the timing for construction of sewer facilities in the Westlake Development and the area along Potomac Creek west of Abel Lake be deferred until development in the area warrants replacement of the Falls Run Interceptor. It is anticipated that development of Westlake and construction of the associated sewer facilities will occur after 2015.

Priority	*3 - Buildout*
Design	*2017*
Construct	*2018*
Design Flow (Upstream/Downstream)	*402 gpm/505 gpm*
Total Project Cost	*$523,000*
Prior Spending	*$0*
Costs in this Plan Period	*$523,000*
Pro Rata Area	*None*

LFR-26: Construct 8-inch gravity main along unnamed tributary to Abel Lake to Upper Potomac Creek PS in vicinity of Cardinal Drive (3,533 feet)

This project includes design and construction of an 8-inch gravity main along unnamed tributary to Abel Lake to Upper Potomac Creek PS in vicinity of Cardinal Drive (3,533 feet). The purpose of the project is to serve future customers in this area. Due to the significant improvements needed for the interceptor along Falls Run which serves this area, it is recommended that the timing for construction of sewer facilities in the Westlake Development and the area along Potomac Creek west of Abel Lake be deferred until development in the area warrants replacement of the Falls Run Interceptor. It is anticipated that development of the area along Potomac Creek west of Abel Lake and construction of the associated sewer facilities will occur after 2015.

Priority	*3 - Buildout*
Design	*2022*
Construct	*2023*
Design Flow (Upstream/Downstream)	*78 gpm/107 gpm*
Total Project Cost	*$481,000*
Prior Spending	*$0*
Costs in this Plan Period	*$481,000*
Pro Rata Area	*None*

LFR-27: Construct 15-inch gravity main along unnamed tributary to Potomac Creek from Centreport Industrial Park to I-95 (6,500 feet)

This project includes design and construction of a 15-inch gravity main along unnamed tributary to Potomac Creek from Centreport Industrial Park to I-95 (6,500 feet). The purpose of the project is to serve future customers in this area.

Priority	*3 - Buildout*
Design	*2019*
Construct	*2020*
Design Flow (Upstream/Downstream)	*630 gpm/1,053 gpm*
Total Project Cost	*$1,253,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,253,000*
Pro Rata Area	*Potomac Creek*

LFR-28: Construct 8-inch gravity main along England Run from vicinity of Trotter Lane to vicinity of Warrenton Road (6,159 feet)

This project includes design and construction of an 8-inch gravity main along England Run from vicinity of Trotter Lane to vicinity of Warrenton Road (6,159 feet). The purpose of the project is to serve future customers in this area. The County has appropriated funds for the segment of gravity sewer (LFR-29) downstream of this segment which is anticipated to occur in the near-term (prior to 2010).

Priority	*2 – Near-term*
Design	*2006*
Construct	*2007*
Design Flow (Upstream/Downstream)	*15 gpm/351 gpm*
Total Project Cost	*$838,000*
Prior Spending	*$0*
Costs in this Plan Period	*$838,000*
Pro Rata Area	*None*

LFR-29: Construct 15-inch gravity main along England Run from England Run PS to 8-inch main along England Run near Trotter Lane (5,427 feet)

This project includes design and construction of a 15-inch gravity main along England Run from England Run PS to 8-inch main along England Run near Trotter Lane (5,427 feet). The purpose of the project is to serve future customers in this area. The County has appropriated funding for this project.

Priority	*5 – Prior Appropriation*
Design	*2005*
Construct	*2005*
Design Flow (Upstream/Downstream)	*376 gpm/1,517 gpm*
Total Project Cost	*$1,046,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,046,000*
Pro Rata Area	*Falls Run*

LFR-30: Construct 12-inch gravity main along unnamed tributary to England Run from England Run PS to Days Inn PS (4,800 feet)

This project includes design and construction of a 12-inch gravity main along unnamed tributary to England Run from England Run PS to Days Inn PS (4,800 feet). The purpose of the project is to convey flows from the upstream interceptors which were constructed to eliminate the Days Inn PS and the Heritage CC PS. The timing for construction of this project should be concurrent with construction of the upstream interceptors (LFR-23) which are dependent on the timing for expansion or major maintenance of the Days Inn PS or Heritage CC PS. It is anticipated that these improvements will not be required in the near-term (prior to 2010).

Priority	*3 - Buildout*
Design	*2020*
Construct	*2021*
Design Flow (Upstream/Downstream)	*315 gpm/726 gpm*
Total Project Cost	*$793,000*
Prior Spending	*$0*
Costs in this Plan Period	*$793,000*
Pro Rata Area	*Falls Run*

LFR-31: Replace 15-inch with 21-inch gravity main along Falls Run from Pennsbury Court to vicinity of Averil Court (3,746 feet)

This project includes replacement of the existing 15-inch with 21-inch gravity main along Falls Run from Pennsbury Court to vicinity of Averil Court (3,746 feet). The existing 15-inch main is roughly 30% to 35% full under near-term (2010) flow conditions and exceeds full flow under buildout conditions.

Priority	*3 - Buildout*
Design	*2017*
Construct	*2018*
Design Flow (Upstream/Downstream)	*2,845 gpm/3,079 gpm*
Total Project Cost	*$1,384,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,384,000*
Pro Rata Area	*Falls Run, Horse Pen, RPR*

LFR-32: Construct 15-inch gravity main along Falls Run from vicinity of Averil Court to vicinity of Holly Corner Road (2,821 feet)

This project includes design and construction of a 15-inch gravity main along Falls Run from vicinity of Averil Court to vicinity of Holly Corner Road (2,821 feet). The purpose of the project is to serve future customers in Westlake. Due to the significant improvements needed for the interceptor along Falls Run which serves this area, it is recommended that the timing for construction of sewer facilities in the Westlake Development and the area along Potomac Creek west of Abel Lake be deferred until development in the area warrants replacement of the Falls Run Interceptor. It is anticipated that development of the area along Potomac Creek west of Abel Lake and construction of the associated sewer facilities will occur after 2015.

Priority	*3 - Buildout*
Design	*2017*
Construct	*2018*
Design Flow (Upstream/Downstream)	*1,482 gpm/1,577 gpm*
Total Project Cost	*$544,000*
Prior Spending	*$0*
Costs in this Plan Period	*$544,000*
Pro Rata Area	*Falls, Run, Horse Pen-RPR*

LFR-33: Construct 8-inch gravity main along Potomac Creek upstream of Upper Potomac Creek PS (2,138 feet)

This project includes design and construction of a 8-inch gravity main along Potomac Creek upstream of Upper Potomac Creek PS (2,138 feet). The purpose of the project is to serve future customers in this area. Due to the significant improvements needed for the interceptor along Falls Run which serves this area, it is recommended that the timing for construction of sewer facilities in the Westlake Development and the area along Potomac Creek west of Abel Lake be deferred until development in the area warrants replacement of the Falls Run Interceptor. It is anticipated that development of the area along Potomac Creek west of Abel Lake and construction of the associated sewer facilities will occur after 2015.

Priority	*3 - Buildout*
Design	*2022*
Construct	*2023*
Design Flow (Upstream/Downstream)	*43 gpm/52 gpm*
Total Project Cost	*$291,000*
Prior Spending	*$0*
Costs in this Plan Period	*$291,000*
Pro Rata Area	*None*

LFR-34: Construct 8-inch gravity main along Potomac Creek upstream of Upper Potomac Creek PS (2,257 feet)

This project includes design and construction of a 8-inch gravity main along Potomac Creek upstream of Upper Potomac Creek PS (2,257 feet). The purpose of the project is to serve future customers in this area. Due to the significant improvements needed for the interceptor along Falls Run which serves this area, it is recommended that the timing for construction of sewer facilities in the Westlake Development and the area along Potomac Creek west of Abel Lake be deferred until development in the area warrants replacement of the Falls Run Interceptor. It is anticipated that development of the area along Potomac Creek west of Abel Lake and construction of the associated sewer facilities will occur after 2015.

Priority	*3 - Buildout*
Design	*2022*
Construct	*2023*
Design Flow (Upstream/Downstream)	*169 gpm/226 gpm*
Total Project Cost	*$307,000*
Prior Spending	*$0*
Costs in this Plan Period	*$307,000*
Pro Rata Area	*None*

LFR-39: Construct 10-inch gravity main along unnamed tributary to Potomac Creek from Leeland Heights PS to proposed Deacon Road Estates PS (2,500 feet)

This project includes design and construction of a 10-inch gravity main along unnamed tributary to Potomac Creek from Leeland Heights PS to proposed Deacon Road Estates PS (2,500 feet). The purpose of the project is to eliminate the Leeland Heights PS and convey flow to the proposed Deacon Road Estates PS. Plans for these improvements have been submitted to the County and the improvements will be implemented in the near-term.

Priority	*1 - Operations*
Design	*2005*
Construct	*2006*
Design Flow (Upstream/Downstream)	*234 gpm/234 gpm*

Total Project Cost	*$292,000*
Prior Spending	*$0*
Costs in this Plan Period	*$292,000*
Pro Rata Area	*None*

LFR-40: Replace 30-inch and 24-inch with 42-inch gravity main along Claiborne Run from Claiborne Run PS to White Oak Road (5,631 feet)

This project includes replacement of the 30-inch and 24-inch with 42-inch gravity main along Claiborne Run from Claiborne Run PS to White Oak Road (5,631 feet). The purpose of this project is to significantly increase the conveyance capacity of the existing interceptor. In addition, the County has identified that the existing interceptor has experienced substantial deterioration and is in poor structural condition. The existing 30-inch main is roughly 40% full under near-term (2010) flow conditions and exceeds full flow under buildout conditions. The County has appropriated funds for replacement of the existing 30-inch interceptor.

Priority	*5 – Prior Appropriation*
Design	*2005*
Construct	*2005*
Design Flow (Upstream/Downstream)	*17,523 gpm/20,585 gpm*
Total Project Cost	*$2,563,000*
Prior Spending	*$0*
Costs in this Plan Period	*$2,563,000*
Pro Rata Area	*Falls Run*

LFR-44: Construct 10-inch gravity main along unnamed tributary to Potomac Creek from Deacon Woods PS to proposed Deacon Road Estates PS (1,200 feet)

This project includes design and construction of a 10-inch gravity main along unnamed tributary to Potomac Creek from Deacon Woods PS to proposed Deacon Road Estates PS (1,200 feet). The purpose of the project is to eliminate the Deacon Woods PS and convey flow to the proposed Deacon Road Estates PS. Plans for these improvements have been submitted to the County and the improvements will be implemented in the near-term.

Priority	*1 - Operations*
Design	*2005*
Construct	*2006*
Design Flow	*143 gpm*
Total Project Cost	*$292,000*
Prior Spending	*$0*
Costs in this Plan Period	*$292,000*
Pro Rata Area	*None*

LFR-46: Construct 8-inch gravity main along unnamed tributary to Potomac Creek in vicinity of Potomac Creek Industrial Park (4,000 feet)

This project includes design and construction of a 8-inch gravity main along unnamed tributary to Potomac Creek in vicinity of Potomac Creek Industrial Park (4,000 feet). The purpose of the project is to serve future customers in the vicinity of the Potomac Creek Industrial Park.

Priority	*3 - Buildout*
Design	*2019*
Construct	*2020*

Design Flow	*207 gpm*
Total Project Cost	*$415,000*
Prior Spending	*$0*
Costs in this Plan Period	*$415,000*
Pro Rata Area	*None*

LFR-47: Construct 15-inch gravity main along Little Falls Run from Little Falls Village PS to Argyle Hills PS (8,500 feet)

This project includes design and construction of a 15-inch gravity main along Little Falls Run from Little Falls Village PS to Argyle Hills PS (8,500 feet). The purpose of the project is to serve future customers east of Little Falls Run and eliminate the Little Falls Village PS; thereby, eliminating pumping and maintenance costs at the pumping station and increasing capacity in the piping downstream of the force main serving the Little Falls Village. The timing for development of the area east of Little Falls Run in the vicinity of the proposed project is anticipated to occur after 2015. Alternatively, DOU could construct additional pumping stations east of Little Falls Run and pump flows west to the existing sewer system. This approach would include replacement of existing sewer piping and expansion of existing pumping stations to accommodate the additional flow from the area east of Little Falls Run.

Priority	*3 - Buildout*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Design Flow (Upstream/Downstream)	*493 gpm/569 gpm*
Total Project Cost	*$1,294,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,294,000*
Pro Rata Area	*Little Falls Run*

LFR-48: Construct 18-inch gravity main along Little Falls Run from Argyle Hills PS to Little Falls Run PS (8,000 feet)

This project includes design and construction of an 18-inch gravity main along Little Falls Run from Argyle Hills PS to Little Falls Run PS (8,000 feet). The purpose of the project is to serve future customers east of Little Falls Run, eliminate the Argyle Hills PS, and convey flows from the proposed upstream interceptor along Little Falls Run to the proposed Little Falls Run PS. Eliminating the Argyle Hills PS eliminates pumping and maintenance costs at the pumping station and increases capacity in the piping downstream of the force main serving the Argyle Hills PS. The timing for development of the area east of Little Falls Run in the vicinity of the proposed project is anticipated to occur after 2015.

Priority	*3 - Buildout*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Design Flow (Upstream/Downstream)	*730 gpm/1,523 gpm*
Total Project Cost	*$1,435,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,435,000*
Pro Rata Area	*Little Falls Run*

LFR-49: Construct 8-inch gravity main along an unnamed tributary from Lake Carroll PS's to Little Falls Run PS (6,800 feet)

This project includes design and construction of an 8-inch gravity main along an unnamed tributary from Lake Carroll PS's to Little Falls Run PS (6,800 feet). The purpose of the project is to serve future customers south of Lake Carroll and eliminate the two pumping stations adjacent to Lake Carroll (PS93 and PS93A). Eliminating the pumping stations eliminates pumping and maintenance costs at the pumping stations and increases capacity in the piping downstream of the force mains serving the pumping stations. The timing for development of the area south of Lake Carroll is anticipated to occur after 2015.

Priority	*3 - Buildout*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Design Flow (Upstream/Downstream)	*58 gpm/194 gpm*
Total Project Cost	*$705,000*
Prior Spending	*$0*
Costs in this Plan Period	*$705,000*
Pro Rata Area	*None*

LFR-50: Construct 12-inch gravity main along unnamed tributary to Potomac Creek from Wagonroad Lane to 15-inch interceptor serving Centreport Industrial Park (4,500 feet)

This project includes design and construction of a 12-inch gravity main along unnamed tributary to Potomac Creek from Wagonroad Lane to 15-inch interceptor serving Centreport Industrial Park (4,500 feet). The purpose of the project is to serve future customers in this area.

Priority	*3 - Buildout*
Design	*2019*
Construct	*2020*
Design Flow (Upstream/Downstream)	*272 gpm/410 gpm*
Total Project Cost	*$744,000*
Prior Spending	*$0*
Costs in this Plan Period	*$744,000*
Pro Rata Area	*Potomac Creek*

7.7.2. Little Falls Run WWTP - Force Mains

LFR-100: Construct 4-inch force main from Rocky Pen Run PS No. 1 to Stafford Lakes PS (3,904 feet)

This project includes design and construction of a 4-inch force main from Rocky Pen Run PS No. 1 to the Stafford Lakes PS (3,904 feet). The purpose of the project is to convey flows from Rocky Pen Run Pumping Station No. 1 which will serve future customers in this area. The timing for construction of this project is dependent on the timing of flows in this area which are anticipated to occur after 2015.

Priority	*3 - Buildout*
Design	*2023*
Construct	*2024*
Design Flow	*218 gpm*
Total Project Cost	*$190,000*
Prior Spending	*$0*
Costs in this Plan Period	*$190, 000*
Pro Rata Area	*None*

LFR-101: Construct 10-inch force main from Westlake PS to Falls Run interceptor (13,397 feet)

This project includes design and construction of a 10-inch force main from the Westlake PS to Falls Run interceptor (13,397 feet). The purpose of the project is to serve future customers in this area. Due to the significant improvements needed for the interceptor along Falls Run which serves this area, it is recommended that the timing for construction of sewer facilities in the Westlake Development and the area along Potomac Creek west of Abel Lake be deferred until development in the area warrants replacement of the Falls Run Interceptor. It is anticipated that development of Westlake and construction of the associated sewer facilities will occur after 2015.

Priority	*3 - Buildout*
Design	*2017*
Construct	*2018*
Design Flow	*1,262 gpm*
Total Project Cost	*$1,063,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,063,000*
Pro Rata Area	*Horse Pen-RPR*

LFR-102: Construct 6-inch force main from Upper Potomac Creek PS No. 1 to 10-inch gravity main connected to Falls Run interceptor (6,632 feet)

This project includes design and construction of a 6-inch force main from Upper Potomac Creek PS No. 1 to 10-inch gravity main connected to Falls Run interceptor (6,632 feet). The purpose of the project is to serve future customers in this area. Due to the significant improvements needed for the interceptor along Falls Run which serves this area, it is recommended that the timing for construction of sewer facilities in the Westlake Development and the area along Potomac Creek west of Abel Lake be deferred until development in the area warrants replacement of the Falls Run Interceptor. It is anticipated that development of the area along Potomac Creek west of Abel Lake and construction of the associated sewer facilities will occur after 2015.

Priority	*3 - Buildout*
Design	*2022*
Construct	*2023*

Design Flow	*226 gpm*
Total Project Cost	*$387,000*
Prior Spending	*$0*
Costs in this Plan Period	*$387,000*
Pro Rata Area	*None*

LFR-103: Construct 12-inch force main from England Run PS to proposed 24-inch main connected to Falls Run interceptor (6,069 feet)

This project includes design and construction of a 12-inch force main from England Run PS to proposed 24-inch main connected to Falls Run interceptor (6,069 feet). The timing for this project should be concurrent with construction of the England Run PS (LFR-204). It is anticipated that development of the area along England Run and construction of the sewer facilities will occur in the near-term (prior to 2010).

Priority	*2 – Near-term*
Design	*2006*
Construct	*2007*
Design Flow	*2,244 gpm*
Total Project Cost	*$992,000*
Prior Spending	*$0*
Costs in this Plan Period	*$992,000*
Pro Rata Area	*Falls Run*

LFR-107: Construct 12-inch force main from Little Falls Run PS (4,000 feet)

This project includes design and construction of a 12-inch force main from Little Falls Run PS (4,000 feet). The timing for this project should be concurrent with construction of the Little Falls Run PS (LFR-205). It is anticipated that development of the area along Little Falls Run and construction of the sewer facilities will occur after 2015.

Priority	*3 - Buildout*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Design Flow	*1,717 gpm*
Total Project Cost	*$343,000*
Prior Spending	*$0*
Costs in this Plan Period	*$343,000*
Pro Rata Area	*Little Falls Run*

LFR-108: Construct 4-inch force main from Rocky Pen Run PS No. 2 to Rocky Pen Run PS No. 1 (2,434 feet)

This project includes design and construction of a 4-inch force main from Rocky Pen Run PS No. 2 to Rocky Pen Run PS No. 1 (2,434 feet). The purpose of the project is to convey flows from Rocky Pen Run Pumping Station No. 2 which will serve future customers in this area. The timing for construction of this project is dependent on the timing of flows in this area which are anticipated to occur after 2015.

Priority	*3 - Buildout*
Design	*2023*
Construct	*2024*
Design Flow	*81 gpm*
Total Project Cost	*$118,000*

Prior Spending	*$0*
Costs in this Plan Period	*$118,000*
Pro Rata Area	*None*

LFR-109: Construct 10-inch force main from Sherwood Forest PS to Little Falls Run WWTP (7,995 feet)

This project includes design and construction of a 10-inch force main from Sherwood Forest PS to Little Falls Run WWTP (7,995 feet). The timing for construction of this project is dependent on the timing of flows in this area which are anticipated to occur after 2015.

Priority	*3 - Buildout*
Design	*2020*
Construct	*2021*
Design Flow	*525 gpm*
Total Project Cost	*$609,000*
Prior Spending	*$0*
Costs in this Plan Period	*$609,000*
Pro Rata Area	*Little Falls Run*

LFR-113: Construct 24-inch force main along Kings Highway from Claiborne Run PS to Little Falls Run WWTP (35,600 feet)

This project includes design and construction of a 24-inch force main along Kings Highway from Claiborne Run PS to Little Falls Run WWTP (35,600 feet). The Claiborne Run PS is currently served by a single 24-inch force main. The two force mains need to have sufficient capacity to convey approximately 30 mgd at buildout. Two 24-inch force mains could carry 30 mgd at approximately 7.4 feet per second which is close to the maximum acceptable velocity. It is anticipated that the existing force main will have sufficient capacity to meet the near-term flows and this project should be constructed in the 2015-2016 timeframe.

Priority	*3 - Buildout*
Design	*2015*
Construct	*2016*
Design Flow	*20,585 gpm (10,292 gpm each)*
Total Project Cost	*$5,306,000*
Prior Spending	*$0*
Costs in this Plan Period	*$5,306,000*
Pro Rata Area	*Claiborne Run, Potomac Creek, Horse Pen-RPR*

LFR-118: Construct 6-inch force main from Deacon Road Estates PS to interceptor along Claiborne Run (5,342 feet)

This project includes design and construction of a 6-inch force main from Deacon Road Estates PS to interceptor along Claiborne Run (5,342 feet). The timing for this project should be concurrent with construction of the Deacon Road Estates PS (LFR-223).

Priority	*1 - Operations*
Design	*2005*
Construct	*2006*
Design Flow	*396 gpm*
Total Project Cost	*$294,000*
Prior Spending	*$0*

Costs in this Plan Period	*$294,000*
Pro Rata Area	*None*

LFR-120: Replace 16-inch with 30-inch force main from Falls Run PS to Claiborne Run interceptor (9,841 feet)

This project includes replacement of the existing 16-inch with 30-inch force main from Falls Run PS to Claiborne Run interceptor (9,841 feet). The existing 16-inch force main from Falls Run PS has sufficient capacity to meet the near-term flows (2010) of approximately 5 mgd. The buildout flow from the Falls Run PS will be approximately 15 mgd which can be carried by a 24-inch main at 7.4 feet per second or by a 30-inch main at 4.7 feet per second. For planning purposes, it is recommended that a 30-inch main be constructed if the 16-inch main is replaced. If a second force main is constructed parallel to the existing 16-inch force main from the Falls Run PS, it is recommended that a 24-inch main be constructed to carry roughly 10 mgd at 4.9 feet per second. In lieu of constructing a second 24-inch force main along Kings Highway from the Claiborne Run PS (LFR-113), DOU may want to consider constructing a 30-inch force main from the Falls Run PS to the Little Falls Run WWTP. Although the cost for construction of the force main would not be significantly different, this option would reduce the size of the expansion required at the Claiborne Run PS (LFR-214), provide additional capacity in the interceptor along Claiborne Run (LFR-42), and eliminate the need for repumping flows at the Claiborne Run PS. The ability to pump flows for this long distance (roughly 40,000 feet) through the force main would need to be evaluated.

Priority	*3 - Buildout*
Design	*2011*
Construct	*2012*
Design Flow	*10,699 gpm*
Total Project Cost	*$1,811,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,811,000*
Pro Rata Area	*Falls Run, Horse Pen-RPR*

LFR-125: Replace 6-inch with 8-inch force main along Morton Road from Hickory Ridge PS to Claiborne Run interceptor (6,725 feet)

This project includes replacement of the existing 6-inch with 8-inch force main along Morton Road from Hickory Ridge PS to Claiborne Run interceptor (6,725 feet). The existing 6-inch force main should have sufficient capacity to meet the near-term flows (prior to 2010).

Priority	*3 - Buildout*
Design	*2011*
Construct	*2012*
Design Flow	*980 gpm*
Total Project Cost	*$436,000*
Prior Spending	*$0*
Costs in this Plan Period	*$436,000*
Pro Rata Area	*Falls Run*

LFR-128: Replace 2-inch with 4-inch force main from PS 91 (367 feet)

This project includes replacement of the 2-inch with 4-inch force main from PS 91 (367 feet). Prior to replacing the existing gravity main, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing force main.

Priority	*7 – Flow Monitoring*
Design	*2014*
Construct	*2014*
Design Flow	*146 gpm*
Total Project Cost	*$18,000*
Prior Spending	*$0*
Costs in this Plan Period	*$18,000*
Pro Rata Area	*Falls Run*

LFR-129: Replace 8-inch with 16-inch force main from Potomac Creek PS (9,055 feet)

This project includes replacement of the existing 8-inch with 16-inch force main from Potomac Creek PS (9,055 feet). The purpose of the project is to serve future customers in the area served by the Potomac Creek PS. The capacity of the existing 8-inch force main should be sufficient to meet the near-term flows (2010).

Priority	*3 - Buildout*
Design	*2019*
Construct	*2020*
Design Flow	*3,015 gpm*
Total Project Cost	*$968,000*
Prior Spending	*$0*
Costs in this Plan Period	*$968,000*
Pro Rata Area	*Potomac Creek*

7.7.3. Little Falls Run WWTP - Pumping Stations

LFR-202: Construct Westlake PS to 1.8 mgd

This project includes design and construction of the Westlake PS to 1.8 mgd. The purpose of the project is to serve future customers in this area. Due to the significant improvements needed for the interceptor along Falls Run which serves this area, it is recommended that the timing for construction of sewer facilities in the Westlake Development and the area along Potomac Creek west of Abel Lake be deferred until development in the area warrants replacement of the Falls Run Interceptor. It is anticipated that development of Westlake and construction of the associated sewer facilities will occur after 2015.

Priority	*3 - Buildout*
Design	*2017*
Construct	*2018*
Design Flow	*1,262 gpm*
Total Project Cost	*$636,000*
Prior Spending	*$0*
Costs in this Plan Period	*$636,000*
Pro Rata Area	*Horse Pen-RPR*

LFR-203: Construct Rocky Pen Run PS No. 1 to 0.31 mgd

This project includes design and construction of Rocky Pen Run PS No. 1 to 0.31 mgd. The purpose of the project is to serve future customers in this area. It is anticipated that development of this area will occur after 2015.

Priority	*3 - Buildout*
Design	*2023*
Construct	*2024*
Design Flow	*218 gpm*
Total Project Cost	*$188,000*
Prior Spending	*$0*
Costs in this Plan Period	*$188,000*
Pro Rata Area	*None*

LFR-204: Construct England Run PS at 3.23 mgd

This project includes design and construction of the England Run PS at 3.23 mgd. The purpose of the project is to serve future customers in this area. It is anticipated that development of the area along England Run and construction of the sewer facilities will occur in the near-term (prior to 2010).

Priority	*2 – Near-term*
Design	*2006*
Construct	*2007*
Design Flow	*2,244 gpm*
Total Project Cost	*$1,131,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,131,000*
Pro Rata Area	*Falls Run*

LFR-205: Construct Little Falls Run PS to 2.47 mgd

This project includes design and construction of Little Falls Run PS to 2.47 mgd. The purpose of this project is to pump flow from the interceptors along Little Falls Run and downstream of Lake Carroll to the Little Falls Run WWTP. It is anticipated that the timing for construction of the sewer facilities in this area will be after 2015.

Priority	*3 - Buildout*
Design	*Beyond 2025*
Construct	*Beyond 2025*
Design Flow	*1,717 gpm*
Total Project Cost	*$865,000*
Prior Spending	*$0*
Costs in this Plan Period	*$865,000*
Pro Rata Area	*Little Falls Run*

LFR-206: Construct Sherwood Forest PS to 0.76 mgd

This project includes design and construction of the Sherwood Forest PS to 0.76 mgd. The purpose of this project is to pump flow from the area served by the Sherwood Farms PS to the Little Falls Run WWTP. It is anticipated that the timing for development in this area and construction of sewer facilities will be after 2015.

Priority	*3 - Buildout*
Design	*2020*
Construct	*2021*
Design Flow	*525 gpm*
Total Project Cost	*$378,000*
Prior Spending	*$0*
Costs in this Plan Period	*$378,000*
Pro Rata Area	*Little Falls Run*

LFR-208: Construct Rocky Pen Run PS No. 2 to 0.12 mgd

This project includes design and construction of Rocky Pen Run PS No. 2 to 0.12 mgd. The purpose of the project is to serve future customers in this area. It is anticipated that development of this area will occur after 2015.

Priority	*3 - Buildout*
Design	*2023*
Construct	*2024*
Design Flow	*81 gpm*
Total Project Cost	*$70,000*
Prior Spending	*$0*
Costs in this Plan Period	*$70,000*
Pro Rata Area	*None*

LFR-209: Expand Falls Run PS to 15.5 mgd

This project includes expansion of Falls Run PS from 9.4 mgd to 15.5 mgd. Flow projections and modeling indicate that the pumping station will have sufficient capacity to meet near-term flows. Development of the area along England Run, Westlake, and along Potomac Creek west of Abel Lake will dictate the timing for expansion of the Falls Run PS. It is anticipated that development of England Run will occur in the near-term and the area of Westlake Development and along Potomac Creek will occur after 2015.

Priority	*3 - Buildout*

Design	*2017*
Construct	*2018*
Design Flow	*10,699 gpm*
Total Project Cost	*$1,204,000*
Prior Spending	*$0*
Costs in this Plan Period	*$1,204,000*
Pro Rata Area	*Falls Run*

LFR-212: Expand Stafford Lakes PS to 1.68 mgd

This project includes expansion of the Stafford Lakes PS from 0.72 mgd to 1.68 mgd. Flow projections and modeling indicate that the pumping station will have sufficient capacity to meet near-term flows. The timing for expansion of the Stafford Lakes PS will be driven by the timing for development of the area served by Rocky Pen Run PS Nos. 1 and 2. It is anticipated that this area will be developed after 2015.

Priority	*3 - Buildout*
Design	*2016*
Construct	*2017*
Design Flow	*1,168 gpm*
Total Project Cost	*$481,000*
Prior Spending	*$0*
Costs in this Plan Period	*$481,000*
Pro Rata Area	*Horsepen-RPR*

LFR-214: Expand Claiborne Run PS to 29.7 mgd

This project includes expansion of the Claiborne Run PS from 8.1 mgd to approximately 18 mgd in 2007-2008 and from 18 mgd to approximately 30 mgd after 2025. Flow projections and modeling indicate that the existing pumping station will have insufficient capacity to meet near-term flows.

Priority	*2 – Near-term*
Design	*2007/Beyond 2025*
Construct	*2008/Beyond 2025*
Design Flow	*20,585 gpm*
Total Project Cost	*$4,316,000*
Prior Spending	*$0*
Costs in this Plan Period	*$4,316,000*
Pro Rata Area	*Claiborne Run, Potomac Creek, Horse Pen-RPR*

LFR-215: Expand Hickory Ridge PS to 1.41 mgd

This project includes expansion of the Hickory Ridge PS from 0.33 mgd to 1.41 mgd. Flow projections and modeling indicate that the pumping station will have insufficient capacity to meet near-term flows.

Priority	*2 – Near-term*
Design	*2009*
Construct	*2010*
Design Flow	*980 gpm*
Total Project Cost	*$540,000*
Prior Spending	*$0*
Costs in this Plan Period	*$540,000*
Pro Rata Area	*Claiborne Run*

LFR-217: Expand Stratford Place PS to 0.39 mgd

This project includes expansion of PS91 from 0.12 mgd to 0.39 mgd. This pumping station serves an area that is partially developed and is served by public sewer. Flow projections and modeling indicate that the pumping station will have insufficient capacity to meet near-term flows. Prior to expanding the existing pumping station, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing pumping station.

Priority	*7 – Flow Monitoring*
Design	*2014*
Construct	*2014*
Design Flow	*268 gpm*
Total Project Cost	*$163,000*
Prior Spending	*$0*
Costs in this Plan Period	*$163,000*
Pro Rata Area	*Little Falls Run*

LFR-221: Expand Boscobel Woods PS to 0.14 mgd

This project includes expansion of Boscobel Woods PS from 0.12 mgd to 0.14 mgd. This pumping station serves an area that is partially developed and is served by public sewer. Flow projections and modeling indicate that the pumping station will have insufficient capacity to meet near-term flows. Prior to expanding the existing pumping station, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing pumping station.

Priority	*7 – Flow Monitoring*
Design	*2014*
Construct	*2014*
Design Flow	*100 gpm*
Total Project Cost	*$17,000*
Prior Spending	*$0*
Costs in this Plan Period	*$17,000*
Pro Rata Area	*Claiborne Run*

LFR-222: Construct Upper Potomac Creek PS to 0.33 mgd

This project includes design and construction of the Upper Potomac Creek PS to 0.33 mgd. The purpose of the project is to serve future customers in this area. Due to the significant improvements needed for the interceptor along Falls Run which serves this area, it is recommended that the timing for construction of sewer facilities in Westlake and the area along Potomac Creek west of Abel Lake be deferred until development in the area warrants replacement of the Falls Run Interceptor. It is anticipated that development of the area along Potomac Creek west of Abel Lake and construction of the associated sewer facilities will occur after 2015.

Priority	*3 - Buildout*
Design	*2022*
Construct	*2023*
Design Flow	*226 gpm*
Total Project Cost	*$195,000*
Prior Spending	*$0*
Costs in this Plan Period	*$195,000*
Pro Rata Area	*None*

LFR-223: Construct Deacon Road Estates PS to 0.57 mgd
This project includes design and construction of the Deacon Road Estates PS to 0.57 mgd. The purpose of the project is to eliminate the Deacon Woods PS and Leeland Heights PS and convey flow to the proposed Deacon Road Estates PS. Plans for these improvements have been submitted to the County and the improvements will be implemented in the near-term.

Priority	*1 - Operations*
Design	*2005*
Construct	*2006*
Design Flow	*396 gpm*
Total Project Cost	*$343,000*
Prior Spending	*$0*
Costs in this Plan Period	*$343,000*
Pro Rata Area	*None*

LFR-224: Expand PS90 to 0.68 mgd
This project includes expansion of PS90 from 0.58 mgd to 0.68 mgd. This pumping station serves an area that is partially developed and is served by public sewer. Flow projections and modeling indicate that the pumping station will have insufficient capacity to meet near-term flows. Prior to expanding the existing pumping station, it is recommended that flow monitoring and/or sewer modeling be performed over a 10-year period to assess the available capacity remaining in the existing pumping station.

Priority	*7 – Flow Monitoring*
Design	*2014*
Construct	*2014*
Design Flow	*470 gpm*
Total Project Cost	*$60,000*
Prior Spending	*$0*
Costs in this Plan Period	*$60,000*
Pro Rata Area	*Falls Run*

LFR-226: Expand Potomac Creek PS to 4.34 mgd
This project includes expansion of the Potomac Creek PS from 0.63 mgd to 4.34 mgd. Flow projections and modeling indicate that the pumping station capacity will approximate the near-term flow in 2010. It is anticipated that a significant portion of the projected flow for this area will occur after 2015.

Priority	*3 - Buildout*
Design	*2019*
Construct	*2020*
Design Flow	*3,015 gpm*
Total Project Cost	*$742,000*
Prior Spending	*$0*
Costs in this Plan Period	*$742,000*
Pro Rata Area	*Potomac Creek*

Appendix A

Near-term Sewer Flows (2010)

DATE: 8/27/04

WWTP	Name	Parcel	Type	Units	Acres	Flow (GPD)	Flow (GPM)	Manhole
Aquia	Airport		Comm.		36	21,240	14.75	PS-45
Aquia	Alta Courthouse APTS	30-29	Resid.	370		104,784	72.77	41-001
Aquia	Alta Courthouse APTS	30-29	Resid.	8		2,266	1.57	41-005
Aquia	Alta Courthouse APTS	30-29	Resid.	20		5,664	3.93	41-006
Aquia	Alta Courthouse TH	30-29	Resid.	145		41,064	28.52	41-006
Aquia	Apple Grove	19-42	Resid.	21		5,947	4.13	49-098
Aquia	Armstrong Tract	28-9	Comm.		150	88,500	61.46	49-621
Aquia	Augustine Central	28-94	Resid.	740		209,568	145.53	58-512
Aquia	Austin Ridge 6C,7,8B		Resid.	104		29,453	20.45	40-1122
Aquia	Austin Ridge elementary		INS					40-1122
Aquia	Austin Ridge 8A		Resid.	57		16,142	11.21	40-1332
Aquia	Austin Ridge Commercial		Comm.		17	10,030	6.97	40-1122
Aquia	Austin Ridge Commercial		Comm.		33	19,470	13.52	43-101
Aquia	Azalea Woods	30-14A	Resid.	84		23,789	16.52	Pump St. to
								40-588
Aquia	Belanders	30-95	Resid.	29		8,213	5.70	40-0523
Aquia	Belanders	30-95	Resid.	57		16,142	11.21	40-0592
Aquia	Bells Run	30-13	Resid.	125		35,400	24.58	40-0508
								40-1108
Aquia	Berkshire	29-3	Resid.	66		18,691	12.98	40-2082
Aquia	Berkshire	29-3	Resid.	5		1,416	0.98	40-2081
Aquia	Brentsmill	21-145	Resid.	160		45,312	31.47	10-308
Aquia	Centreport	37-30	Comm.		25	14,750	10.24	PS-45
Aquia	Centreport	37-30	Comm.		70	41,300	28.68	70-124
Aquia	Centreport	37-30	Comm.		37	21,830	15.16	70-135
Aquia	Churchill Meadows	18-40	Resid.	91		25,771	17.90	water only
Aquia	Claiborne Manor	19-15	Resid.	28		7,930	5.51	49-705
Aquia	Colonial Port		Resid.	21		5,947	4.13	16-108
Aquia	Dogwoods	12-24	Resid.	148		41,914	29.11	12-212B
Aquia	Ellison Estates	38-49	Resid.	16		4,531	3.15	water only
Aquia	Embrey Mill		Resid.	523		148,248	102.95	40-1124 & 40-2016
Aquia	Embrey Mill	29-53	Resid.	1,285		363,773	252.62	
Aquia	Fritter Park	13-67	Resid.		84	49,560	34.42	12-103
Aquia	Hamlets of Widewater	23-1	Resid.	251		71,083		water only
Aquia	Hills of Aquia	21-50	Resid.	268		75,898	52.71	10-093
Aquia	Hillside Terrace	12A-1	Resid.	49		13,877	9.64	12-152
Aquia	Lake Estates		Resid.	28		6,720	4.67	70-135
Aquia	Liberty Knolls	29-18	Resid.	102		28,886	20.06	58-522
Aquia	Manors @ Greenridge	30-151	Resid.	34		9,629	6.69	40-0677
Aquia	North Stafford Business		Comm.		43	25,370	17.62	49-099
Aquia	Popular Estates	27-1	Resid.	132		37,382	25.96	water only
Aquia	Port Aquia	21-65	Resid.	250		70,800	49.17	10-202
Aquia	Quantico		Wholesale			395,400		40-0197
Aquia	Rolling Meadows	28-38	Resid.	18		5,098	3.54	water only

								DATE: 8/27/04
WWTP	**Name**	**Parcel**	**Type**	**Units**	**Acres**	**Flow (GPD)**	**Flow (GPM)**	**Manhole**
Aquia	Roseville		Resid.	150		42,480	29.50	water only
Aquia	Seasons Landing	30-114	Resid.	183		51,826	35.99	134
Aquia	Seneca Ridge	38-55	Resid.	30		8,496	5.90	45-107
Aquia	Shelton Shop Center	19-23A	Comm.		36	21,240	14.75	40-0247
Aquia	Smith Lake Estates		Resid.	29		8,213	5.70	38-141
Aquia	Somerset Landing	30-119	Resid.	126		35,683	24.78	Tamarlane
								to 40-0801
Aquia	St Georges	19-12	Resid.	178		50,410	35.01	49-1113
								PIPE
Aquia	St Georges Est. sec. 8	19-12	Resid.	10		2,832	1.97	49-790
Aquia	Stafford MarketPlace	21-8	Comm.		40	23,600	16.39	RT95 crossing to 10-512
Aquia	Stowe of Amyclae	28-116	Resid.	143		40,498	28.12	40-2084
								40-2094
Aquia	Summerwood	30-145	Resid.	41		11,611	8.06	56-216
Aquia	Summit Ridge	30-100	Resid.	31		8,779	6.10	40-0517A
Aquia	Tamarlane	30-116	Resid.	84		23,789	16.52	40-0801
Aquia	The Glens	27-17	Resid.	173		48,994	34.02	water only
Aquia	The Reserve	39-48	Resid.	24		6,797	4.72	water only
Aquia	Turney Estates	27-8	Resid.	22		6,230	4.33	water only
Aquia	Widewater Hills	22-21	Resid.	69		19,541		water only
Aquia	Widewater Village	21-100A	Resid.	354		100,253	69.62	10-134
Aquia	Woodstream	21-8	Resid.	494		139,901	97.15	RT95 crossing to 10-512
Aquia	Woodstream	21-8	Resid.	256		72,499	50.35	10-219
Aquia	**Total**			**7,632**	**571**	**2,892,458**		
LFR	Cambridge Crossing	45-205	Resid.	56		15,859	11.01	60-1997
LFR	Cannon Ridge	27-32A	Resid.	27		7,646	5.31	82-150
LFR	Cannon Ridge	27-32A	Resid.	173		48,994	34.02	82-147
LFR	Carriage Hills @ Falls Run	53A-1-30	Resid.	115		32,568	22.62	60-0119
LFR	Carriage Hills @ Falls Run	53A-1-31	Resid.	32		9,062	6.29	60-0113
LFR	Celebrate Virginia	44-7	Comm.		24.1	14,219	9.87	86
LFR	Celebrate Virginia	44-7	Comm.		11.23	6,626	4.60	88
LFR	Celebrate Virginia	52-1	Comm.		5.33	3,145	2.18	116
LFR	Celebrate Virginia	44-76	Resid.	6		1,699	1.18	86
LFR	Celebrate Virginia	44-89	Resid.	1,371		388,267	269.63	116
LFR	Cranewood	45-281	Resid.	10		2,832	1.97	80-2014
LFR	Crescent Valley	54C-1-30	Resid.	44		12,461	8.65	82-145
LFR	Crescent Valley	54C-1-30	Resid.	4		1,133	0.79	82-123
LFR	Deacon Woods Estates	54-129	Resid.	70		19,824	13.77	89-0412
LFR	England Run 2A		Resid.	80		22,656	15.73	60-0192
LFR	England Run 3A		Resid.	103		29,170	20.26	60-0186
LFR	England Run 3B		Resid.	89		25,205	17.50	60-0186

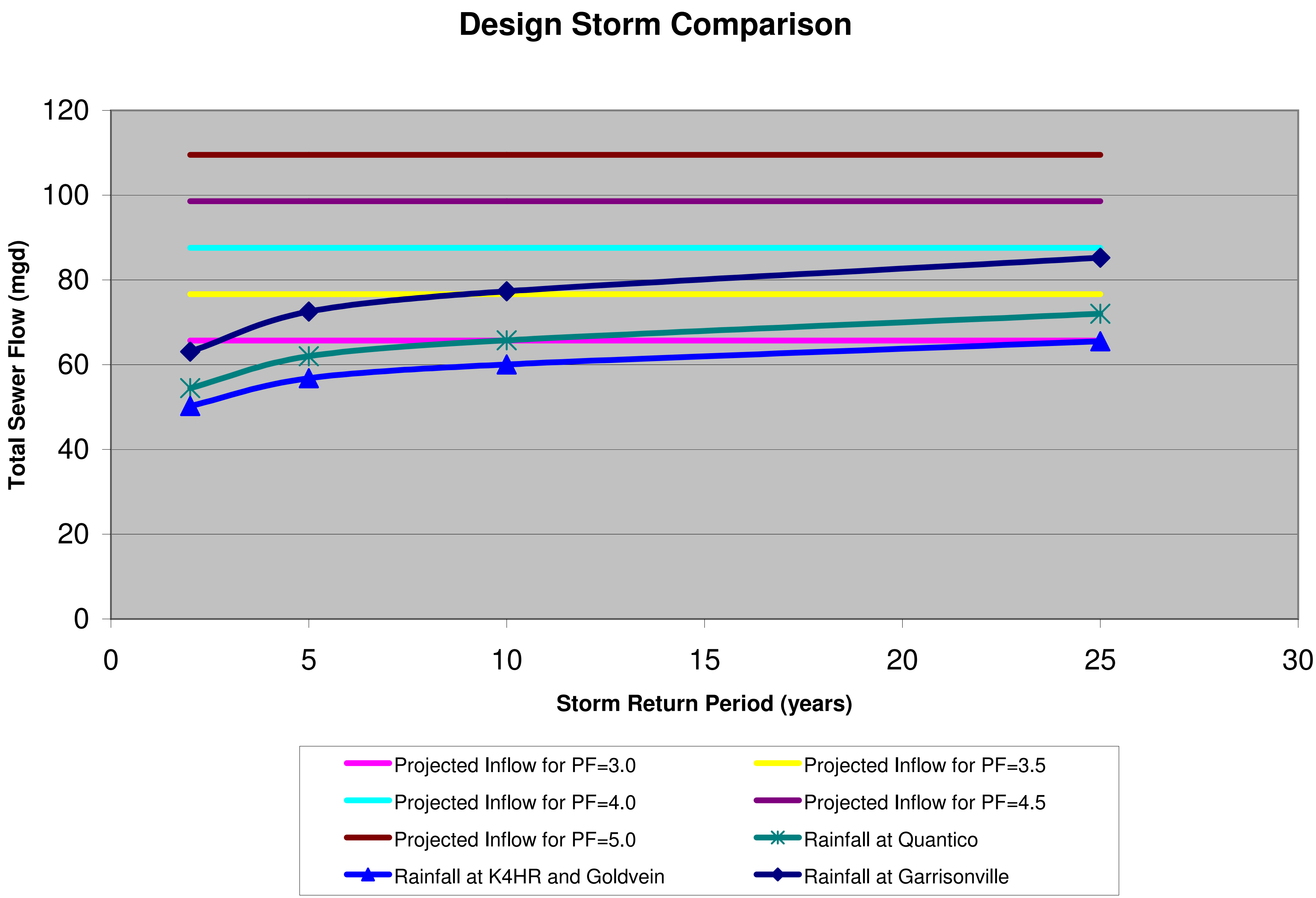
DATE: 8/27/04

WWTP	Name	Parcel	Type	Units	Acres	Flow (GPD)	Flow (GPM)	Manhole
LFR	England Run 4		Resid.	55		15,576	10.82	60-0407
LFR	England Run 4		Resid.	135		38,232	26.55	60-0402
LFR	England Run 7		Resid.	14		3,965	2.75	60-0407
LFR	England Run 7		Resid.	46		13,027	9.05	60-0402
LFR	England Run 8		Resid.	11		3,115	2.16	60-0425
LFR	England Run 9		Resid.	139		39,365	27.34	60-0407
LFR	England Run 18		Resid.	56		15,859	11.01	60-0211
LFR	England Run 19		Resid.	123		34,834	24.19	60-0211
LFR	England Run 20		Resid.	2		566	0.39	60-0208
LFR	England Run 33		Resid.	119		33,701	23.40	60-0425
LFR	Fitzhugh	55-14	Resid.	23		6,514	4.52	99-110
LFR	Fitzhugh	55-14	Resid.	22		6,230	4.33	99-123
LFR	Gollahon	46-4			28.8	16,992	11.80	70-116
LFR	Gollahon	46-2	Comm.					
LFR	Heather Hills V	54-20	Resid.	57		16,142	11.21	80-0151
LFR	Heather Hills IV	54-20	Resid.	31		8,779	6.10	80-1367
LFR	Hickory Ridge	45-56	Resid.	107		30,302	21.04	PS-86 to
								80-0164
LFR	High School	55-157D	INS					82-180
LFR	Middle School	46-106	INS					89-0412
LFR	Landsberry Park	54C-1-25	Resid.	18		5,098	3.54	82-123
LFR	Leeland ES	46-93F	INS					80-1004
LFR	Leeland Station	46-48	Resid.	34		9,629	6.69	80-0209
LFR	Leeland Station		Comm.		8.74	5,157	3.58	80-0209
LFR	Leeland Station		Resid.	706		199,939	138.85	80-0154
LFR	Leeland Station		Comm.		11.26	6,643	4.61	80-0154
LFR	Lynnbrooke Commons	53-76	Resid.	17				60-0907
LFR	Oaks @ Ferry Farm	54-156	Resid.	15		4,248	2.95	83-1094
LFR	Oaks @ Ferry Farm	54-156	Resid.	12		3,398	2.36	94-119
LFR	Oaks of Highland Homes	54A-1D-19	Resid.	10		2,832	1.97	80-0728
LFR	Oaks of Highland Homes	54A-1D-19	Resid.	14		3,965	2.75	80-1393
LFR	Rappahannock Landing	53-1H	Resid.	413		116,962	81.22	61-131
LFR	Rappahannock Landing	53-1H	Resid.	279		79,013	54.87	61-126
LFR	Riverside Business Park	45-31	Comm.		20	11,800	8.19	62-508
LFR	Scotsdale Estates	55-63A	Resid.	16		4,531	3.15	75-834
LFR	Sherwood Forest		Comm.		300	177,000	122.92	
LFR	Shimco Property	38-23F	Comm.		35	20,650	14.34	70-702
LFR	Stafford Lake Village 6-10	43-73	Resid.	331		93,739	65.10	64-101
LFR	Stafford Lake Village 11,12		Resid.	222		62,870	43.66	PS to GS to 64-101
LFR	Stafford Lake Village 13,14		Resid.	183		51,826	35.99	PS to GS to 64-101
LFR	Staffordshire	36-65	Resid.	168		47,578	33.04	214

								DATE: 8/27/04
WWTP	**Name**	**Parcel**	**Type**	**Units**	**Acres**	**Flow (GPD)**	**Flow (GPM)**	**Manhole**
LFR	Staffordshire	36-65	Resid.	168		47,578	33.04	218
LFR	Stratford Place		Resid.	139		39,365	27.34	75-9001
LFR	Sydney Hastings	45-92			15	8,850	6.15	60-0146
LFR	Taylor Bott Industrial Park	38-15	Comm.		91	53,690	37.28	70-738
LFR	Towering Oaks	54-60	Resid.	20		5,664	3.93	80-3008
LFR	Town & County Marketplace	54-114	Comm.		15	8,850	6.15	83-960
LFR	Westlake	35-20	Resid.	796		225,427	156.55	134
LFR	Woodland Woods	54-116	Resid.	9		2,549	1.77	80-0542
LFR	Forbes Landing	46-27	Resid.	90		25,488	17.70	80-0167
LFR	**Total**			**6,780**	**565**	**2,248,903**	**1561.74**	
Notes:								
1. Flow Allowances								
	Residential (GPD)	240	per HU					
	Commercial (GPD)	500	per Acre					
	Unaccountable water	18%						
2. Peaking factor		1.6						

Appendix B

Computation of R-values for Storm Events


Design Storm Comparison
120
100
80
60
40
20
0
Total Sewer Flow (mgd)
0
5
10
15
20
25
30
Storm Return Period (years)
Projected Inflow for PF=3.0
Projected Inflow for PF=3.5
Projected Inflow for PF=4.0
Projected Inflow for PF=4.5
Projected Inflow for PF=5.0
Rainfall at Quantico
Rainfall at K4HR and Goldvein
Rainfall at Garrisonville

Garrisonville Rainfall Data

PS Name	Monitoring Point	Date	Inflow Total (gal)	Rainfall Total (in)	Area (Acres)	Conversion Factor	R	Average R
Austin Run	AR-PS 40 (6)	8/28-30/2002	2,358,480	2.01	4227	27152.4	0.0102	
Country Ridge	AR-PS 47 (11)	8/28-30/2002	75,763	2.01	272	27152.4	0.0051	
Garrisonville Est.	AR-PS 49 (20)	8/28-30/2002	592,830	2.01	786	27152.4	0.0138	
Upper Accokeek	AR-PS 58 (44)	8/28-30/2002	43,192	2.01	119	27152.4	0.0066	
	AR - Basin 08	4/7-12/2003	3,867,895	1.41	3548	27152.4	0.0285	
	AR - Basin 08	4/18-19/03	1,437,143	0.71	3548	27152.4	0.0210	
Aquia at Bridge	AH-PS 20 (4)	8/28-29/02	304,850	2.01	338	27152.4	0.0165	
Aquia at Dewey	AH-PS 31 (2)	8/28-29/02	155,190	2.01	767	27152.4	0.0037	
Falls Run	R-PS 60 (18)	8/28-29/02	829,538	2.01	1180	27152.4	0.0129	
								0.0132

Storm Event Return Interval (Years)	Inflow Total (gal)	24-Hour Rainfall Total from IDF Curve (in)	Area of Future Urban Service Area (Acres)	Conversion Factor	R
2	**41,160,054**	**3.12**	**36943**	**27152.4**	**0.0132**
5	**50,658,528**	**3.84**	**36943**	**27152.4**	**0.0132**
10	**55,407,765**	**4.2**	**36943**	**27152.4**	**0.0132**
25	**63,323,160**	**4.8**	**36943**	**27152.4**	**0.0132**

Storm Event Return Interval (Years) and Peaking Factors	Average Dry Weather Flow at Buildout (mgd)	Rainfall-Dependent I/I (mgd)	Peak Wet Weather Flow at Buildout (mgd)	Graph Values for PWWF at Buildout using a Peaking Factor of 3.0 (mgd)	Graph Values for PWWF at Buildout using a Peaking Factor of 3.5 (mgd)	Graph Values for PWWF at Buildout using a Peaking Factor of 4.0 (mgd)	Graph Values for PWWF at Buildout using a Peaking Factor of 4.5 (mgd)	Graph Values for PWWF at Buildout using a Peaking Factor of 5.0 (mgd)
2	22	41.16	63	66	77	88	99	110
5	22	50.66	73	66	77	88	99	110
10	22	55.41	77	66	77	88	99	110
25	22	63.32	85	66	77	88	99	110
PF=3	22	43.80	66					
PF=3.5	22	54.75	77					
PF=4	22	65.70	88					
PF=4.5	22	76.65	99					
PF=5	22	87.60	110					

Quantico Rainfall Data

PS Name	Monitoring Point	Date	Inflow Total (gal)	Rainfall Total (in)	Area (Acres)	Conversion Factor	R	Average R
Austin Run	AR-PS 40 (6)	8/28-30/2002	2,358,480	3.02	4227	27152.4	0.0068	
Country Ridge	AR-PS 47 (11)	8/28-30/2002	75,763	3.02	272	27152.4	0.0034	
Garrisonville Est.	AR-PS 49 (20)	8/28-30/2002	592,830	3.02	786	27152.4	0.0092	
Upper Accokeek	AR-PS 58 (44)	8/28-30/2002	43,192	3.02	119	27152.4	0.0044	
	AR - Basin 08	4/7-12/2003	3,867,895	1.72	3548	27152.4	0.0233	
	AR - Basin 08	4/18-19/03	1,437,143	0.61	3548	27152.4	0.0245	
Aquia at Bridge	AH-PS 20 (4)	8/28-29/02	304,850	3.02	338	27152.4	0.0110	
Aquia at Dewey	AH-PS 31 (2)	8/28-29/02	155,190	3.02	767	27152.4	0.0025	
Falls Run	R-PS 60 (18)	8/28-29/02	829,538	3.02	1180	27152.4	0.0086	
								0.0104

Storm Event Return Interval (Years)	Inflow Total (gal)	24-Hour Rainfall Total from IDF Curve (in)	Area of Future Urban Service Area (Acres)	Conversion Factor	R
2	**32,563,418**	**3.12**	**36943**	**27152.4**	**0.0104**
5	**40,078,053**	**3.84**	**36943**	**27152.4**	**0.0104**
10	**43,835,370**	**4.2**	**36943**	**27152.4**	**0.0104**
25	**50,097,566**	**4.8**	**36943**	**27152.4**	**0.0104**

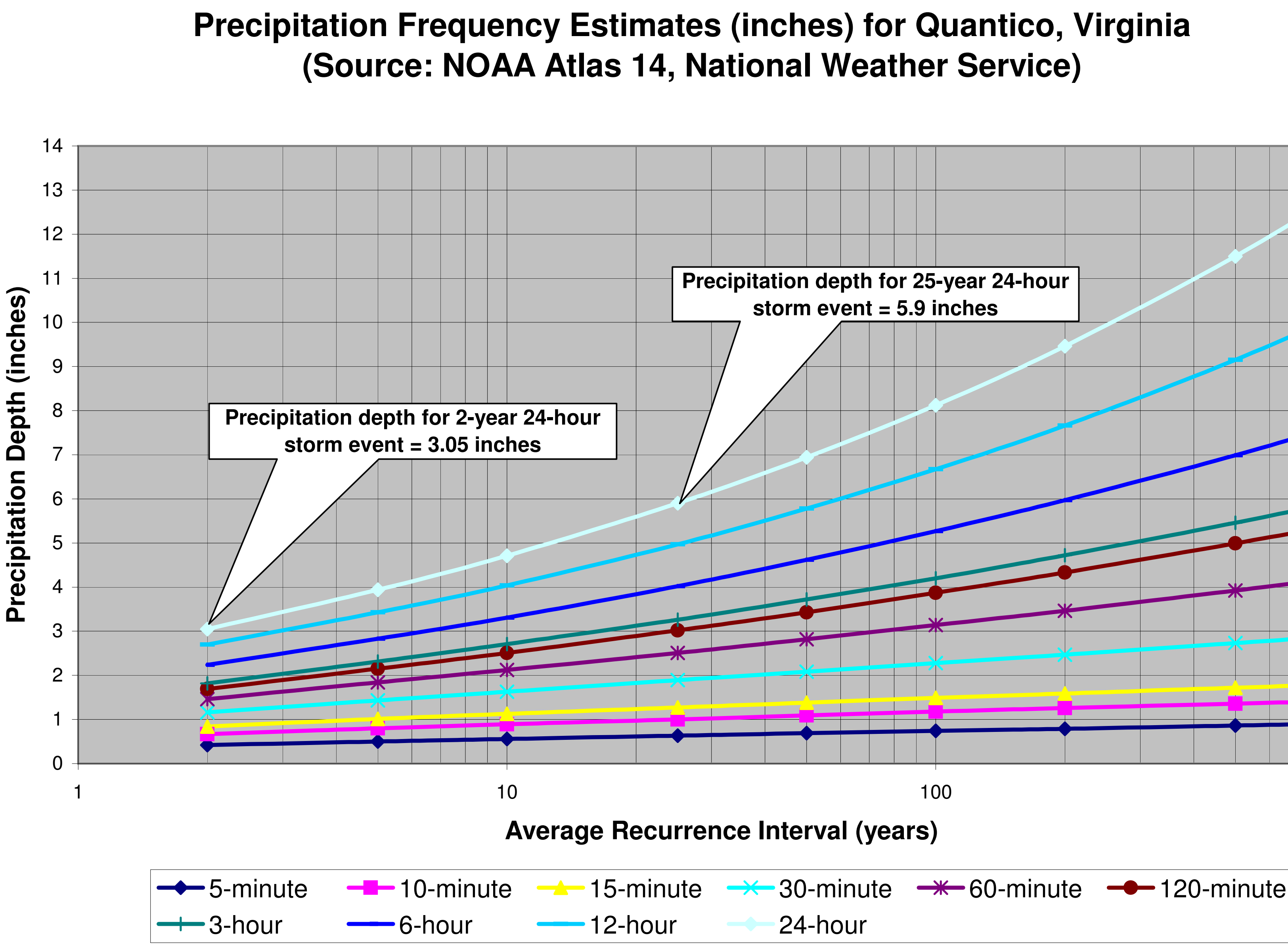
Storm Event Return Interval (Years) and Peaking Factors	Average Dry Weather Flow at Buildout (mgd)	Rainfall-Dependent I/I (mgd)	Peak Wet Weather Flow at Buildout (mgd)	Graph Values for PWWF at Buildout using a Peaking Factor of 3.0 (mgd)	Graph Values for PWWF at Buildout using a Peaking Factor of 3.5 (mgd)	Graph Values for PWWF at Buildout using a Peaking Factor of 4.0 (mgd)	Graph Values for PWWF at Buildout using a Peaking Factor of 4.5 (mgd)	Graph Values for PWWF at Buildout using a Peaking Factor of 5.0 (mgd)
2	22	32.56	54	66	77	88	99	110
5	22	40.08	62	66	77	88	99	110
10	22	43.84	66	66	77	88	99	110
25	22	50.10	72	66	77	88	99	110
PF=3	22	43.80	66					
PF=3.5	22	54.75	77					
PF=4	22	65.70	88					
PF=4.5	22	76.65	99					
PF=5	22	87.60	110					

K4HR/Goldvien VA Rainfall Data

PS Name	Monitoring Point	Date	Inflow Total (gal)	Rainfall Total (in)	Area (Acres)	Conversion Factor	R	Average R	
Austin Run	AR-PS 40 (6)	8/28-30/2002	2,358,480	3.47	4227	27152.4	0.0059		
Country Ridge	AR-PS 47 (11)	8/28-30/2002	75,763	3.47	272	27152.4	0.0030		
Garrisonville Est.	AR-PS 49 (20)	8/28-30/2002	592,830	3.47	786	27152.4	0.0080		
Upper Accokeek	AR-PS 58 (44)	8/28-30/2002	43,192	3.47	119	27152.4	0.0038		
									K4HR Rainfall Data
	AR - Basin 08	4/7-12/2003	3,867,895	1.92	3548	27152.4	0.0209		K4HR Rainfall Data
	AR - Basin 08	4/18-19/03	1,437,143	0.72	3548	27152.4	0.0207		K4HR Rainfall Data
Aquia at Bridge	AH-PS 20 (4)	8/28-29/02	304,850	3.47	338	27152.4	0.0096		
Aquia at Dewey	AH-PS 31 (2)	8/28-29/02	155,190	3.47	767	27152.4	0.0021		
Falls Run	R-PS 60 (18)	8/28-29/02	829,538	3.47	1180	27152.4	0.0075		
								0.0091	

Storm Event Return Interval (Years)	Inflow Total (gal)	24-Hour Rainfall Total from IDF Curve (in)	Area of Future Urban Service Area (Acres)	Conversion Factor	R
2	**28,351,218**	**3.12**	**36943**	**27152.4**	**0.0091**
5	**34,893,806**	**3.84**	**36943**	**27152.4**	**0.0091**
10	**38,165,101**	**4.2**	**36943**	**27152.4**	**0.0091**
25	**43,617,258**	**4.8**	**36943**	**27152.4**	**0.0091**

Storm Event Return Interval (Years) and Peaking Factors	Average Dry Weather Flow at Buildout (mgd)	Rainfall-Dependent I/I (mgd)	Peak Wet Weather Flow at Buildout (mgd)	Graph Values for PWWF at Buildout using a Peaking Factor of 3.0 (mgd)	Graph Values for PWWF at Buildout using a Peaking Factor of 3.5 (mgd)	Graph Values for PWWF at Buildout using a Peaking Factor of 4.0 (mgd)	Graph Values for PWWF at Buildout using a Peaking Factor of 4.5 (mgd)	Graph Values for PWWF at Buildout using a Peaking Factor of 5.0 (mgd)
2	22	28.35	50	66	77	88	99	110
5	22	34.89	57	66	77	88	99	110
10	22	38.17	60	66	77	88	99	110
25	22	43.62	66	66	77	88	99	110
PF=3.0	22	43.80	66					
PF=3.5	22	54.75	77					
PF=4.0	22	65.70	88					
PF=4.5	22	76.65	99					
PF=5.0	22	87.60	110					



Precipitation Frequency Estimates (inches) for Quantico, Virginia
(Source: NOAA Atlas 14, National Weather Service)
Precipitation depth for 25-year 24-hour storm event = 5.9 inches
Precipitation depth for 2-year 24-hour storm event = 3.05 inches
Precipitation Depth (inches)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
1
10
100
1000
Average Recurrence Interval (years)
5-minute
10-minute
15-minute
30-minute
60-minute
120-minute
3-hour
6-hour
12-hour
24-hour

* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr
2	0.42	0.67	0.84	1.16	1.46	1.69	1.82	2.24	2.7	3.05
5	0.5	0.8	1.01	1.43	1.84	2.15	2.31	2.83	3.43	3.94
10	0.56	0.89	1.13	1.63	2.12	2.51	2.71	3.31	4.04	4.71
25	0.63	1	1.27	1.89	2.51	3.02	3.26	4.02	4.97	5.9
50	0.69	1.09	1.38	2.08	2.82	3.43	3.72	4.62	5.78	6.94
100	0.74	1.18	1.49	2.28	3.14	3.87	4.2	5.27	6.67	8.12
200	0.79	1.26	1.59	2.47	3.46	4.33	4.72	5.97	7.66	9.46
500	0.86	1.36	1.72	2.73	3.92	4.99	5.46	6.99	9.15	11.5
1000	0.92	1.44	1.81	2.93	4.28	5.52	6.07	7.85	10.43	13.28

TECHNICAL MEMORANDUM 9

Resource Recovery

(Text to be prepared by Stafford County DOU)

TECHNICAL MEMORANDUM 10

Summary of Water Planning and Design Criteria

Prepared for:	Stafford County Department of Utilities
Prepare by:	O'Brien & Gere
Date:	November 2004

This technical memorandum is one of a series being prepared for the Water and Sewer Master Plan project. The purpose of this technical memorandum is to summarize the approach and water planning and design criteria used in the Master Plan and identify the location in the Master Plan for the supporting documentation.

This document contains the following sections:

Terminology, Definitions and Glossary

Average Dry Weather Flow (ADWF) – ADWF consists of average daily sewage flows and groundwater infiltration (GWI). ADWF is the average flow that occurs on a daily basis with no evident reaction to rainfall.

C-factor – A measure of the interior roughness of a pipe**.**

Diurnal Demand or Flow – Fluctuation of water demands or wastewater flows over a 24-hour period.

Effective Storage – Effective storage for each storage facility is determined by establishing the level in each tank above which all points in the water system can be served at 20 psi or higher (based on peak hour or maximum day plus fire flow).

Equalization Storage – The storage of peaking flows to prevent overflows from the sewer collection and conveyance systems.

Groundwater Infiltration (GWI) – Groundwater that infiltrates pipeline and manhole defects located below the ground surface. Groundwater infiltration is separate and distinguished from inflow resulting from storm events. Infiltration is a steady 24-hour flow that usually varies during the year in relation to the groundwater levels above the sewers. Infiltration rates are normally estimated from wastewater flows measured in the sewers during the early morning hours when water use is at a minimum and the flow is essentially infiltration.

H2OMAP Water – H2OMAP Water is a computer model used for modeling the Department of Utilities' water system under various demand conditions.

H2OMAP Sewer – H2OMAP Sewer is a computer model used for modeling the Department of Utilities' sewer system under various flow conditions.

Inflow – Drainage that enters the collection system through illegal or permitted connections, such as catch basins, downspouts, area drains and manhole covers. Inflow is separate and distinguished from infiltration. The inflow rate can be determined from the flow hydrographs recorded with flow meters by subtracting the normal dry weather flow and the infiltration from the measured flowrate.

Infiltration/Inflow (I/I) – The wastewater component caused by rainfall-dependent infiltration/inflow (RDI/I) and groundwater infiltration (GWI).

Maximum Day Demand – The one day in the year when the consumption is the highest.

Maximum Hour Demand – The one hour in the year when water consumption is the highest.

Node – A junction of two or more pipes, commonly representing a point where pipe characteristics change.

Peak Dry Weather Flow (PDWF) – PDWF consists of peak sewage flows plus GWI. PDWF is the highest measured hourly flow that occurs on a dry weather day.

Peak Wet Weather Flow (PWWF) – PWWF consists of ADWF plus RDI/I. PWWF is the highest measured hourly flow that occurs during wet weather.

Peak Factor – Peak factor is PWWF/ADWF.

Pressure Reducing Valve (PRV) – A valve that will maintain a specified downstream pressure.

Pressure Zone – A network of water pipes having a common static hydraulic grade line. Pressure zones are separated by closed valves, pressure regulating valves, pumping stations, and reservoirs.

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Service Area – The area served by the water distribution or wastewater collection system.

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Tributary Area – The tributary area of a sewage system consists of all areas that contribute flow to the sewer by gravity and/or force main discharges.

ADD	Average Day Demand
ADWF	Average Dry Weather Flow
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EPS	Extended Period Simulation
ft	Feet
FY	Fiscal Year
gpcpd	Gallons per Capita per Day
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gpdidm	Gallons per Day per Inch Diameter – Mile
GWI	Groundwater Infiltration
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ICR	Information Collection Rule
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L	Liter
MCL	Maximum Contaminant Level
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MG	Million Gallons
MGD	Million Gallons Per Day
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WTP	Water Treatment Plant
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1.0. Water Demand Factors

Note to Reader: Refer to Technical Memorandum 2 (Water Demands) for a detailed discussion of the information in this section.

Average daily per capita water demand 80 gpd

Table 1. Water duties

Land Use	Water Duties (gpd/acre)	Reference
Suburban Residential	500	Refer to Technical Memorandum 2 – Water Demands (Section 3.0)
Urban Residential	1300	
Rural Residential	80	
Agricultural	40	
Commercial	750	
Office	500	
Light Industrial	500	
Heavy Industrial	2000	
Institutional	500	

Notes:
1. Resource Protection Areas and Parks are excluded from the calculation of acreage of land use.
2. Water duties are to be used with 100% development of the land use category (i.e., acreage includes existing and future road corridors, on-site stormwater facilities, etc.) and are intended to reflect demands and flows that will be generated by the actual amount of acreage developed.
3. Water duties represent water demands and an allowance for unaccounted-for water (i.e., "water produced" minus "water sold") is not included in the water duties (note that a global allowance of 15% was added to the water demands for modeling water system facilities).

2.0. Water Demands

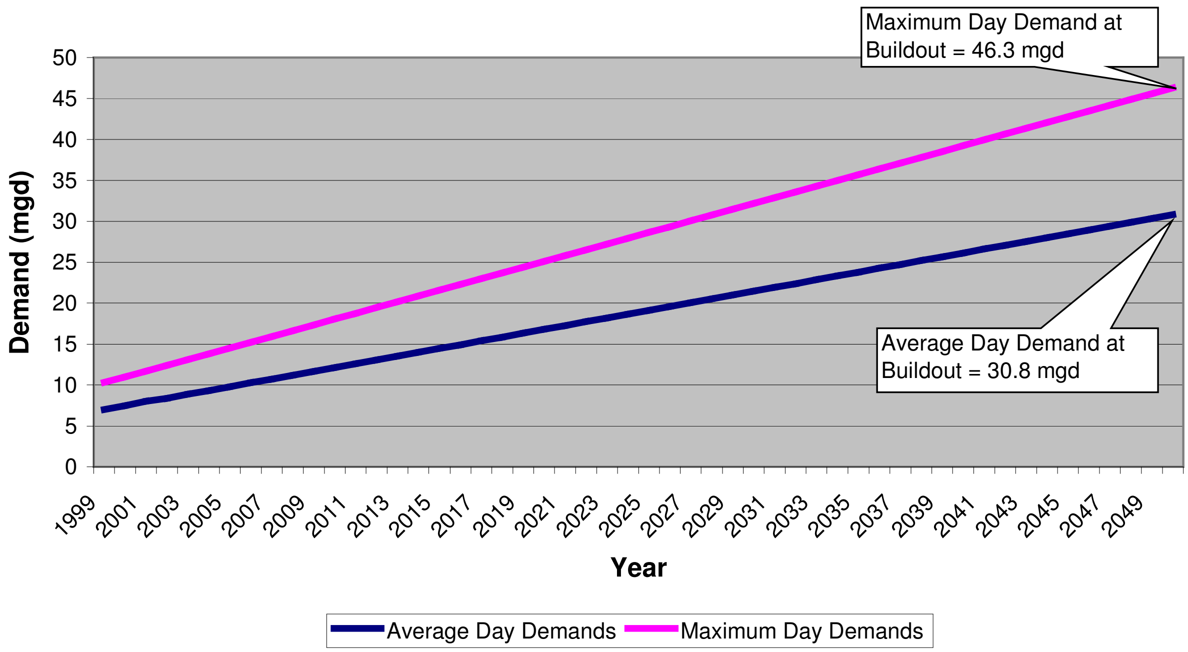
Note to Reader: Refer to Technical Memorandum 2 (Water Demands) for a detailed discussion of the information in this section.

Water systems are required to supply flow at rates that fluctuate over a wide range from day-to-day and hour-to-hour. Rates most important to planning, design and operation of a water system are annual average day, maximum (peak) day, maximum (peak) hour, and maximum day plus fire flow.

- Annual average day demand is the total volume of water delivered to the system in a given year divided by the number of days in the year.
- Maximum (peak) day demand is the largest quantity of water supplied to the system on any given day of the year.
- Maximum (peak) hour demand is the highest rate of flow for any hour in a year.
- Maximum day plus fire flow considers the possibility of a fire event under maximum day demand conditions.

The peak day factor (maximum day demand/average day demand) for 2002 was 1.67. Peaking factors will drop as the system continues to expand through the planning period. Average day water demands are expected to increase from 8.4 mgd (2003) to roughly 30.8 mgd under buildout (2050) conditions. During the same period, the maximum day water demands are expected to increase from approximately 13 mgd (2003) to 46 mgd at buildout (2050) based on a peaking factor of 1.5 times the average day demand. The water demands are shown in Figure 1.

Figure 1: Projected Water Demands



3.0. Water System Planning and Design Criteria

Note to Reader: Refer to Section 5.0 of Technical Memorandum 5 (Water Facilities) for a detailed discussion of the information in this section.

3.1. Overview of Water System Planning and Design Criteria

For this Master Plan, DOU's planning and design criteria for waterworks facilities is summarized as follows:

- Water treatment facilities shall be adequate to provide the maximum day water demand.
- Water booster pumping stations shall be adequate to pump the maximum day water demand. While pumping stations are typically sized for maximum day demands, it may be desirable to size pumping facilities for peak hour demands (or a portion of peak hour demands) if the pumping station serves a pressure zone with a single storage tank that must be taken out-of-service for maintenance. It is generally desirable to provide at least two storage tanks per pressure zone to simplify operation of the pumping facilities when a tank is taken out-of-service. The Virginia Department of Health's (VDH) "Waterworks Regulations" require that each pumping station shall have at least two pumping units. Pumps should have sufficient capacity so that if any one pump is out-of-service (firm capacity) the remaining units shall be capable of providing the maximum day demand.
- Pipelines are sized for the following:

  - The largest of maximum hour flow, maximum day flow plus fire flow, or replenishment flow. Fire flow requirements are a primary factor affecting the sizing of piping in the water distribution system (6-inch and 8-inch mains).
  - An allowable velocity of 5 ft/sec.
  - An allowable headloss of 2-5 feet/1,000 feet of pipeline.
- Maximum water pressures at the service connections were 120 psi.
- Minimum water pressures were 45 psi at the service connection at maximum day demand rates and water storage tanks at 10 feet below overflow levels, and 20 psi at the service connection based on the greater of maximum hour or maximum day plus fire flow demand condition.
- Pressure fluctuation was limited to 20-30 psi.
- Pressure zone layout was based on the minimum pressure established by the highest ground elevation that can be supplied, and the maximum pressure established by the lowest ground elevation.
- Pressure regulating valves were proposed with a minimum pressure differential of 10 psi for small valves (6-inch and smaller) and 5 psi for large valves (8-inch and larger). The maximum velocity allowed through the valve is typically 15-20 feet/sec.
- Looping was considered to provide a higher level of reliability (i.e., if one source is out-of-service to the area, supply can be provided from a second source).
- Pipe materials generally accepted include ductile iron, steel, concrete, and polyvinyl chloride (plastic or PVC). PVC is usually used for smaller diameter piping.

3.2. Fire Flow Requirements

Fire flow requirements are typically dependent on the land use and vary by community. Stafford County's fire flow requirements are shown in Table 2.

Table 2. Fire flow requirements

	Land Use		
Source	**Residential**	**Commercial**	**Industrial**
Stafford County	1000 – 2500 gpm	2500 gpm	2500 gpm

3.3. Storage Criteria

According to the VDH "Waterworks Regulations", the minimum acceptable effective finished water storage for domestic purposes must be greater than 200 gallons per equivalent residential connection at minimum pressure (this essentially equates to one-half of the annual average day demand). For this Master Plan, the volume of storage needed will be equal to one-half of the annual average day demand.

4.0. Summary

The approach and criteria outlined in this technical memorandum are based on sound engineering and give reasonable projections of future water demands and design demand conditions. One of the key benefits of this approach is that the conservatism in the approach resides in the latter stages of the planning process.

TECHNICAL MEMORANDUM 11

Summary of Sewer Planning and Design Criteria

Prepared for:	Stafford County Department of Utilities
Prepare by:	O'Brien & Gere
Date:	November 2004

This technical memorandum is one of a series being prepared for the Water and Sewer Master Plan project. The purpose of this technical memorandum is to summarize the approach and sewer planning and design criteria used in the Master Plan and identify the location in the Master Plan for the supporting documentation.

This document contains the following sections:

Terminology, Definitions and Glossary

Average Dry Weather Flow (ADWF) – ADWF consists of average daily sewage flows and groundwater infiltration (GWI). ADWF is the average flow that occurs on a daily basis with no evident reaction to rainfall.

C-factor – A measure of the interior roughness of a pipe**.**

Diurnal Demand or Flow – Fluctuation of water demands or wastewater flows over a 24-hour period.

Effective Storage – Effective storage for each storage facility is determined by establishing the level in each tank above which all points in the water system can be served at 20 psi or higher (based on peak hour or maximum day plus fire flow).

Equalization Storage – The storage of peaking flows to prevent overflows from the sewer collection and conveyance systems.

Groundwater Infiltration (GWI) – Groundwater that infiltrates pipeline and manhole defects located below the ground surface. Groundwater infiltration is separate and distinguished from inflow resulting from storm events. Infiltration is a steady 24-hour flow that usually varies during the year in relation to the groundwater levels above the sewers. Infiltration rates are normally estimated from wastewater flows measured in the sewers during the early morning hours when water use is at a minimum and the flow is essentially infiltration.

H2OMAP Water – H2OMAP Water is a computer model used for modeling the Department of Utilities' water system under various demand conditions.

H2OMAP Sewer – H2OMAP Sewer is a computer model used for modeling the Department of Utilities' sewer system under various flow conditions.

Inflow – Drainage that enters the collection system through illegal or permitted connections, such as catch basins, downspouts, area drains and manhole covers. Inflow is separate and distinguished from infiltration. The inflow rate can be determined from the flow hydrographs recorded with flow meters by subtracting the normal dry weather flow and the infiltration from the measured flowrate.

Infiltration/Inflow (I/I) – The wastewater component caused by rainfall-dependent infiltration/inflow (RDI/I) and groundwater infiltration (GWI).

Maximum Day Demand – The one day in the year when the consumption is the highest.

Maximum Hour Demand – The one hour in the year when water consumption is the highest.

Node – A junction of two or more pipes, commonly representing a point where pipe characteristics change.

Peak Dry Weather Flow (PDWF) – PDWF consists of peak sewage flows plus GWI. PDWF is the highest measured hourly flow that occurs on a dry weather day.

Peak Wet Weather Flow (PWWF) – PWWF consists of ADWF plus RDI/I. PWWF is the highest measured hourly flow that occurs during wet weather.

Peak Factor – Peak factor is PWWF/ADWF.

Pressure Reducing Valve (PRV) – A valve that will maintain a specified downstream pressure.

Pressure Zone – A network of water pipes having a common static hydraulic grade line. Pressure zones are separated from other zones by closed valves, pressure regulating valves, pumping stations, and reservoirs.

Rainfall-Dependent Infiltration/Inflow (RDI/I) – RDI/I consists of rainfall that enters the collection system through direct connections (roof leaders, manholes, etc.) and causes an almost immediate increase in wastewater flow.

Service Area – The area served by the water distribution or wastewater collection system.

Steady State Simulation – A network model solution for a single point in time.

Tributary Area – The tributary area of a sewage system consists of all areas that contribute flow to the sewer by gravity and/or force main discharges.

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1.0. Water Demand and Wastewater Flow Factors

Note to Reader: Refer to Technical Memorandum 2 (Water Demands) and Section 4.0 of Technical Memorandum 6 (Rainfall/Flow Monitoring Program) for a detailed discussion of the information in this section.

Average daily per capita water demand	80 gpd
Average daily per capita sewage flow (80% of per capita water demand)	64 gpd

Table 1. Water duties and sewage flow factors

Land Use	Water Duties (gpd/acre)	Sewage Flow Factors at 80% of Water Duties (gpd/acre)	Reference
Suburban Residential	500	400	Refer to Technical Memorandum 2 – Water Demands
Urban Residential	1300	1040	
Rural Residential	80	64	
Agricultural	40	32	
Commercial	750	600	
Office	500	400	
Light Industrial	500	400	
Heavy Industrial	2000	1600	
Institutional	500	400	

Notes:
1. Resource Protection Areas and Parks are excluded from the calculation of acreage of land use.
2. Water duties are to be used with 100% development of the land use category (i.e., acreage includes existing and future road corridors, on-site stormwater facilities, etc.) and are intended to reflect demands and flows that will be generated by the actual amount of acreage developed.
3. Water duties represent water demands and an allowance for unaccounted-for water (i.e., "water produced" minus "water sold") is not included in the water duties (note that a global allowance of 15% was included for modeling water system facilities).
4. Sewage flow factors do not include an allowance for groundwater infiltration (GWI) during dry weather conditions (i.e., not rainfall-dependent flow) or inflow from storm events. These allowances are to be added to the sanitary base flow (derived from sewage flow factors) to obtain the design flow.

Return flows to the sanitary sewer system (sanitary base flows) were computed by defining the tributary area of each manhole and applying the sewage flow factors in Table 1 to the land uses within the tributary area.

2.0. Sewer Loads for Dry Weather Conditions

Note to Reader: Refer to Section 3.0 of Technical Memorandum 7 (Development and Calibration of H2OMAP Sewer Hydraulic Model) for a detailed discussion of the information in this section.

Average Dry Weather Flow (ADWF) = Sanitary Base Flow + Groundwater Infiltration (GWI)

Where:

Average Dry Weather Flow (ADWF) is the average flow that occurs in sanitary sewers on a daily basis with no evident reaction to rainfall.

Sanitary base flow equals the average daily water demand based on water duties presented in the previous section multiplied by 80% which is an estimate of the customer water demand that is returned to the sanitary sewer.

Groundwater infiltration (GWI) is an allowance that is added to the sanitary base flow (derived from sewage flow factors) to obtain the dry weather flow. GWI represents flow that is separate and distinguished from inflow resulting from storm events during wet weather conditions. The allowance used in this Master Plan for GWI is estimated to be 500 gpd/inch diameter-mile (gpdidm).

3.0. Sewer Loads for Wet Weather Conditions

Note to Reader: Refer to Section 5.0 of Technical Memorandum 7 (Development and Calibration of H2OMAP Sewer Hydraulic Model) and Section 4.0 of Technical Memorandum 8 (Wastewater Collection, Pumping and Conveyance Facilities) for detailed discussions of the information in this section.

Peak Wet Weather Flow (PWWF) = Average Dry Weather Flow (ADWF) + Rainfall-Dependent I/I (RDI/I)

Where:

Peak Wet Weather Flow (PWWF) equals the peak hourly flow during wet weather conditions.

Average Dry Weather Flow (ADWF) is the average flow that occurs in sanitary sewers on a daily basis with no evident reaction to rainfall.

Rainfall-Dependent I/I consists of rainfall that enters the collection system through direct connections (roof leaders, manholes, etc.) and causes an almost immediate increase in wastewater flows. RDI/I data from an August 2002 storm event (2-year return interval) was used for sewer model calibration. For the August 28, 2002 storm event, peaking factors at various pumping stations ranged from 2.6 to 3.7 (i.e., peak hourly flows were 2.6 to 3.7 times greater than the average dry weather flow for that period). The weighted (based on number of upstream manholes) peaking factor for the overall sewer system was approximately 2.8 for the August 28, 2002 storm event.

Additional flow monitoring information is needed to accurately predict the response of the sewer system to larger storm events with varying characteristics (i.e., intensity, duration, and volume). To define the design flow conditions for the sewer system, the equation presented above was modified as follows:

Peak Wet Weather Flow (PWWF) = Average Dry Weather Flow (ADWF) x Peak Factor

The peak factor is equal to the PWWF/ADWF. Based on the results of the August 2002 storm event, industry guidelines, and anticipated regulatory requirements, a peak factor of 3.5 is used to derive the peak wet weather flow for a storm event with an estimated 25-year recurrence interval and 24-hour duration. In the H2OMAP Sewer model, the peak factor of 3.5 is multiplied by the sanitary base flow at each manhole in the sewer system and the GWI component (500 gpdidm) is subsequently added to the computed manhole flow as the flow is routed through the downstream sewer piping.

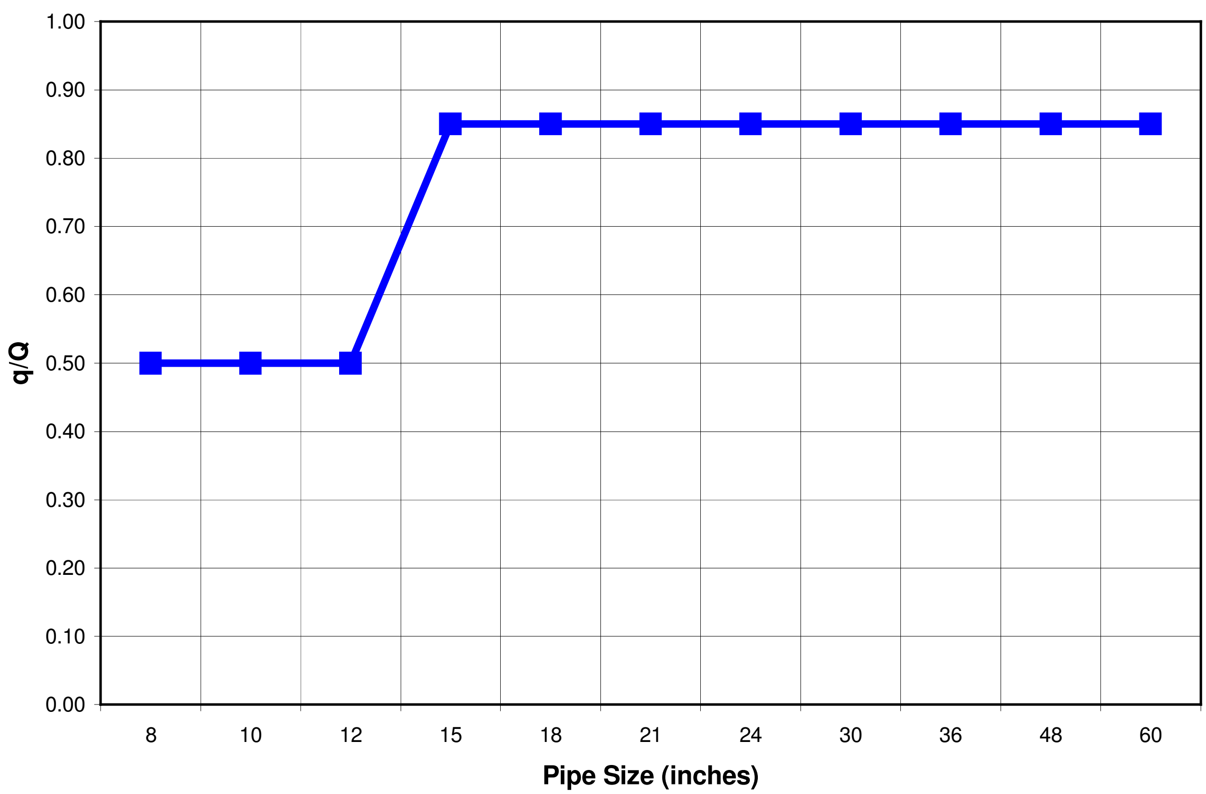
4.0. Sewer System Design Criteria

Note to Reader: Refer to Section 5.0 of Technical Memorandum 8 (Wastewater Collection, Pumping and Conveyance Facilities) for a detailed discussion of the information in this section.

"n" value	0.013 for all pipe materials
Minimum Velocity	2.25 ft/sec
Maximum Velocity	15 ft/sec
Minimum Depth of Cover	3 feet
Maximum Depth of Cover	20 feet

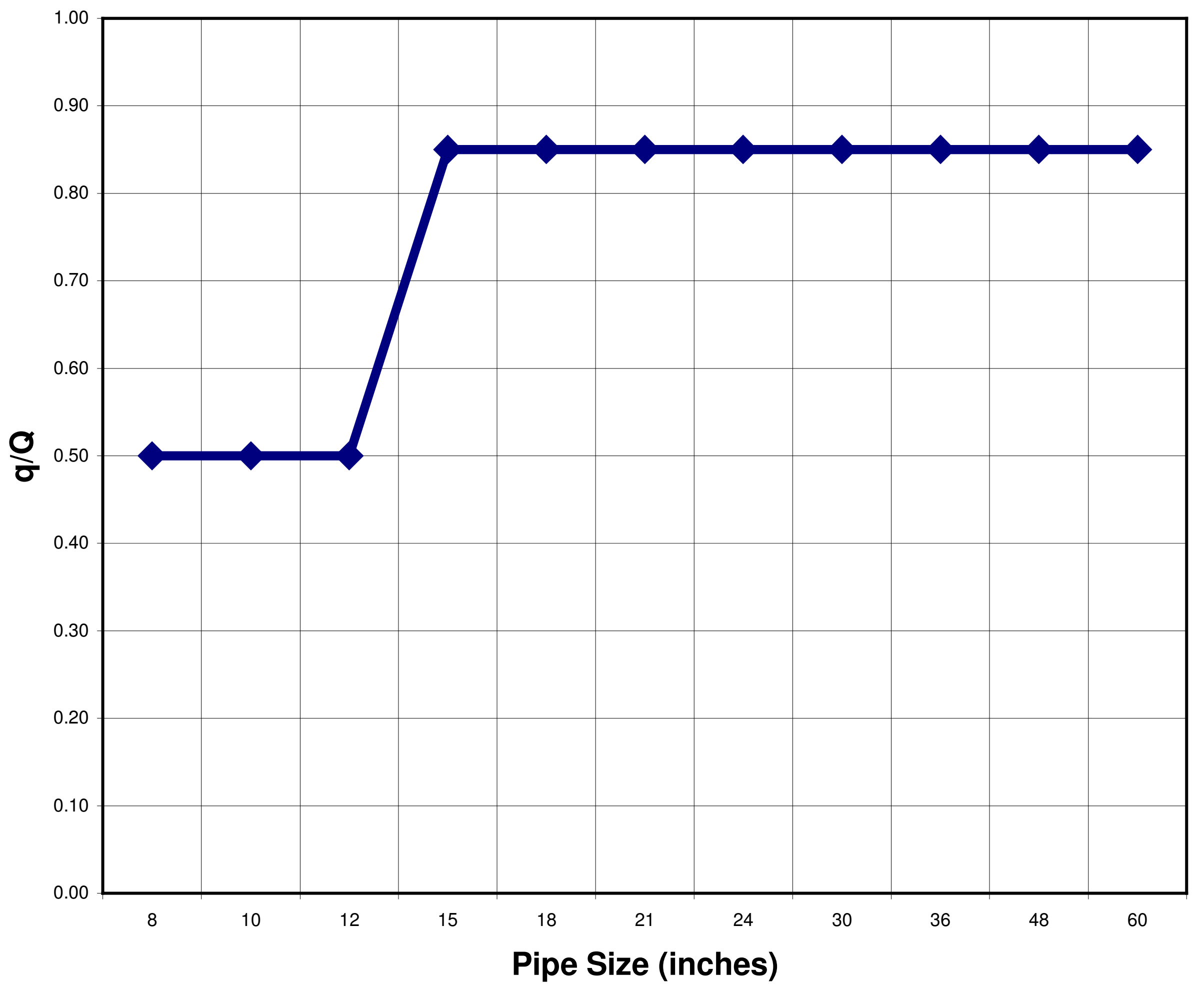
The H2OMAP Sewer model includes analysis and design criteria curves which are effective and efficient tools that can be used under steady-state conditions for evaluating the capabilities of the existing system and sizing improvements to the sewer system. Analysis and design criteria curves have been developed for this study to define the “threshold” values at which point capacity enhancement measures for pipelines within the sanitary sewer system should be evaluated and rehabilitated or replaced. The partial flow-to-full flow ratios used to develop the analysis criteria curve are shown in Figure 1 and were less conservative for the large diameter sewer pipelines (15 inches and larger in diameter). The q/Q ratio of 0.85 (d/D ratio of 0.75) for the large diameter pipelines reflects the desire to maximize flow in the existing interceptor sewers. The q/Q ratio for small diameter pipelines maintains some reserve capacity and reflects the uncertainty in the spatial distribution of sewer loads served by the smaller piping in the sewer system. By applying relatively conservative q/Q ratios for the analysis curve, pipelines will be identified prior to reaching full capacity and thus reduce the likelihood of surcharge and/or overflow conditions. It should be noted that existing pipelines that exceeded the design criteria and were less than full through buildout conditions (q/Q less than 1.0) were not recommended for replacement. Rather, these pipelines were flagged for future investigation and possible flow monitoring during the planning period.

Figure 1 - Analysis Criteria Curve



The design criteria curve is used for designing the relief or replacement pipelines when the capacity of the existing pipelines has been exceeded as defined by the analysis criteria curve. In general, the design criteria curve generally reflects the desire to limit the possibility of requiring additional improvements in the near-term planning period. The design criteria curve values used in this study are shown in Figure 2.

Figure 2 - Design Criteria Curve



5.0. Summary

The approach and criteria outlined in this technical memorandum are based on sound engineering and give reasonable projections of future sewer flows and design flow conditions. One of the key benefits of this approach is that the conservatism in the approach resides in the latter stages of the planning process - application of the peak factor and the analysis and design curves used for sizing of piping.

TECHNICAL MEMORANDUM 12

Cost Estimates

Prepared for: Stafford County Department of Utilities
Prepare by: O'Brien & Gere
Date: November 2004

This technical memorandum is one of a series being prepared for the Stafford County Water and Sewer Master Plan project. The purpose of this technical memorandum is to document the unit cost basis and assumptions used for estimating construction costs for water treatment, pumping, and storage facilities; water transmission and distribution system piping; and wastewater collection system piping and pumping facilities. Project costs to be incorporated into the County's capital improvements program will be generated by adding allowances to the estimated construction costs.

The cost estimates generated for this study are termed "budget" estimates and are appropriate for the level of detail associated with concept level planning. Budget level estimates are made without detailed engineering data or information on site-specific conditions (e.g., final pipeline alignments, aesthetics, etc.). The intended use of these estimates is for developing budgets for inclusion in the County's capital program. Budget level estimates are considered accurate within +30% and –15%.

This document contains the following sections:

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Inflow – Drainage that enters the collection system through illegal or permitted connections, such as catch basins, downspouts, area drains and manhole covers. Inflow is separate and distinguished from infiltration. The inflow rate can be determined from the flow hydrographs recorded with flow meters by subtracting the normal dry weather flow and the infiltration from the measured flowrate.

Infiltration/Inflow (I/I) – The wastewater component caused by rainfall-dependent infiltration/inflow (RDI/I) and groundwater infiltration (GWI).

Maximum Day Demand **–** The one day in the year when the consumption is the highest.

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Node – A junction of two or more pipes, commonly representing a point where pipe characteristics change.

Peak Dry Weather Flow (PDWF) – PDWF consists of peak sewage flows plus GWI. PDWF is the highest measured hourly flow that occurs on a dry weather day.

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Peak Factor – Peak factor is PWWF/ADWF.

Pressure Reducing Valve (PRV) – A valve that will maintain a specified downstream pressure.

Pressure Zone – A network of water pipes having a common static hydraulic grade line. Pressure zones are separated by closed valves, pressure regulating valves, pumping stations, and reservoirs.

Rainfall-Dependent Infiltration/Inflow (RDI/I) – RDI/I consists of rainfall that enters the collection system through direct connections (roof leaders, manholes, etc.) and causes an almost immediate increase in wastewater flow.

Service Area – The area served by the water distribution or wastewater collection system.

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NEPA	National Environmental Policy Act
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WTP	Water Treatment Plant
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Executive Summary

Project cost estimates were developed for water and sewer facility improvements identified in this Master Plan (e.g., water treatment plants, water and sewer pumping stations, and water and sewer system piping). Project cost estimates are intended for use in budget development, wherever site-specific costs are not utilized. The project cost estimates represent typical experience and should be adjusted, where appropriate, to meet special needs.

1.0. Construction Cost Assumptions

The budget level estimates prepared for this study are based on cost curves, previous estimates and historical data from comparable work, estimating guides and handbooks, and local manufacturers' cost data. Cost data has been updated as necessary to reflect current values using Engineering News Record (ENR) indices. Costs are based on August 2003 dollars and an ENR Construction Cost Index for August 2003 (6733).

Cost assumptions of particular note for the water and sewer system follow.

1.1. Water Treatment Plants

The construction and project costs for expanding Smith Lake WTP or Abel Lake WTP are based on the findings from the facility assessments performed in this study. Construction costs for a new water treatment plant at the proposed Rocky Pen Run Reservoir are based on $1.25/gallon of installed capacity before allowances.

1.2. Water Pumping Stations

Where appropriate, the construction and project costs for expanding pumping stations are based on the findings from the facility assessments performed in this study. Construction costs for new pumping stations were based on installed capacity before allowances as shown in Table 1.

Table 1. Construction costs for water pumping stations

Installed Capacity	Construction Cost
Less than 1 mgd	$300,000
2 mgd	$500,000
5 mgd	$1,000,000
10 mgd and above	$0.15/gallon

1.3. Water Storage Facilities

Finished water storage facilities proposed for this study were elevated storage tanks. Construction costs for elevated water storage were estimated based on $1.25/gallon for 0.5 MG to 2 MG facilities.

1.4. Water Pipelines

The costs for installing pipe are dependent on ground conditions (land use) and geography (roads, rivers, railroad crossings, etc.). For example, installing pipe in an urban setting is typically more costly than installation in a rural area for a variety of reasons. The reasons include a greater likelihood of construction in the roadway instead of the right-of-way, a higher potential for conflict with other utilities and greater difficulty in maintaining traffic.

Higher costs for pipes apply for water mains installed across waterways (streams and rivers), railway, or highway crossings. Costs for tunneling (railroad and highway crossings) were added to the baseline costs for installing water mains.

The unit cost for construction of water pipelines depends on a number of site-specific factors. Higher unit costs ($/ft) are typical for smaller pipelines and for pipelines constructed in areas with a high potential for utility conflicts and traffic disruptions (i.e., areas with a high level of residential and commercial development). Table 2 shows the estimated cost for installation of water pipelines east and west of I-95. The major assumptions follow:

- Costs for pipelines include basic costs, pavement restoration and traffic control.
- Pipelines constructed west of I-95 include 20% allowance for rock excavation.
- Pipelines would be installed in the public rights-of-way.
- The lengths of the crossings for minor, major and interstate roadways were assumed to be 30 feet, 60 feet and 500 feet, respectively.

Detailed alignment studies will be required prior to design and construction.

Table 2. Construction costs for water pipelines

Pipe Diameter (inches)	Open-cut Unit Cost for Piping East of I-95 ($/ft)	Open-cut Unit Cost for Piping West of I-95 ($/ft)	Minor Roadway Crossing ($)	Major Roadway Crossing ($)	Interstate Roadway Crossing ($)
6	34	36	9,660	16,320	200,000
8	40	43	9,840	16,680	200,000
10	47	49	10,020	17,040	200,000
12	53	56	11,700	20,400	200,000
16	66	70	14,310	25,620	200,000
20	79	84	14,670	26,340	200,000
24	92	100	17,280	31,560	200,000
30	111	122	19,320	35,640	225,000
36	130	144	21,360	39,720	250,000
42	149	166	24,900	46,800	300,000
48	169	169	26,940	50,880	325,000

1.5. Sewer Pipelines

The unit cost for gravity sewer pipelines will be dependent on the trench depth and the potential for utility conflicts, maintaining traffic control, and other construction difficulties. Tables 3, 4, and 5 show the unit costs for construction and replacement of proposed gravity sewer pipelines east and west of I-95 for trenches with depths of 3-10 feet, 10-20 feet, and 20-30 feet, respectively. To reflect the potential additional cost for utility conflicts and construction difficulties associated with existing development, a 50 percent increase in the unit cost of construction has been included for replacement pipelines. For this Master Plan, the unit costs for construction and replacement of gravity sewer pipelines were based on trenches with depths of 10-20 feet as shown in Table 4. The major assumptions follow:

- Costs for pipelines include basic costs, pavement restoration and traffic control.
- Pipelines constructed west of I-95 include 20% allowance for rock excavation.
- Pipelines would be installed in the public rights-of-way.
- The lengths of the crossings for minor, major and interstate roadways were assumed to be 30 feet, 60 feet and 500 feet, respectively.

Table 3. Construction and replacement costs for gravity sewer pipelines (3-10 feet depth)

Pipe Diameter (inches)	Open-cut Unit Cost for Piping East of I-95 ($/ft)		Open-Cut Unit Cost for Piping West of I-95 ($/ft)		Minor Roadway Crossing ($)	Major Roadway Crossing ($)	Interstate Roadway Crossing ($)
	Construct	Replace	Construct	Replace			
8	42	63	46	69	9,600	14,000	200,000
10	49	74	52	78	9,750	14,250	200,000
12	55	83	59	89	11,400	17,000	200,000
15	67	101	72	108	13,950	21,250	200,000
18	80	120	86	129	14,250	21,750	200,000
21	95	143	104	156	16,800	26,000	233,000
24	108	162	118	177	18,600	29,000	263,000
30	118	177	131	197	20,250	31,750	290,500
36	129	194	145	218	23,400	37,000	343,000
42	146	219	164	246	25,200	40,000	373,000
48	163	245	184	276	27,000	43,000	403,000

Table 4. Construction and replacement costs for gravity sewer pipelines (10-20 feet depth)

Pipe Diameter (inches)	Open-cut Unit Cost for Piping East of I-95 ($/ft)		Open-Cut Unit Cost for Piping West of I-95 ($/ft)		Minor Roadway Crossing ($)	Major Roadway Crossing ($)	Interstate Roadway Crossing ($)
	Construct	Replace	Construct	Replace			
8	64	96	84	126	9,600	14,000	200,000
10	72	108	93	140	9,750	14,250	200,000
12	80	120	102	153	11,400	17,000	200,000
15	94	141	119	179	13,950	21,250	200,000
18	108	162	136	204	14,250	21,750	200,000
21	123	185	152	228	16,800	26,000	233,000
24	137	206	170	255	18,600	29,000	263,000
30	152	228	189	284	20,250	31,750	290,500
36	167	251	210	315	23,400	37,000	343,000
42	187	281	236	354	25,200	40,000	373,000
48	207	311	262	393	27,000	43,000	403,000

Table 5. Construction and replacement costs for gravity sewer pipelines (20-30 feet depth)

Pipe Diameter (inches)	Open-cut Unit Cost for Piping East of I-95 ($/ft)		Open-Cut Unit Cost for Piping West of I-95 ($/ft)		Minor Roadway Crossing ($)	Major Roadway Crossing ($)	Interstate Roadway Crossing ($)
	Construct	Replace	Construct	Replace			
8	84	126	118	177	9,600	14,000	200,000
10	93	140	130	195	9,750	14,250	200,000
12	102	153	141	212	11,400	17,000	200,000
15	118	177	161	242	13,950	21,250	200,000
18	134	201	181	272	14,250	21,750	200,000
21	151	227	201	302	16,800	26,000	233,000
24	167	251	221	332	18,600	29,000	263,000
30	185	278	248	372	20,250	31,750	290,500
36	204	306	274	411	23,400	37,000	343,000
42	227	341	307	461	25,200	40,000	373,000
48	252	378	340	510	27,000	43,000	403,000

The unit cost for construction of force mains is shown in Table 6. The basic cost assumptions used for water mains in Table 2 apply to force mains.

Table 6. Construction costs for sewer force mains

Pipe Diameter (inches)	Open-cut Unit Cost for Piping East of I-95 ($/ft)	Open-cut Unit Cost for Piping West of I-95 ($/ft)	Minor Roadway Crossing ($)	Major Roadway Crossing ($)	Interstate Roadway Crossing ($)
6	34	36	9,660	16,320	200,000
8	40	43	9,840	16,680	200,000
10	47	49	10,020	17,040	200,000
12	53	56	11,700	20,400	200,000
16	66	70	14,310	25,620	200,000
20	79	84	14,670	26,340	200,000
24	92	100	17,280	31,560	200,000
30	111	122	19,320	35,640	225,000
36	130	144	21,360	39,720	250,000
42	149	166	24,900	46,800	300,000
48	169	169	26,940	50,880	325,000

1.6. Wastewater Pumping Stations

The unit cost for construction of wastewater pumping stations ranges from $0.15/gallon to $0.60/gallon as shown in Table 7.

Table 7. Construction costs for wastewater pumping stations

Installed Capacity	Construction Cost
Less than 0.5 mgd	$300,000
1 mgd	$500,000
2 mgd	$700,000
5 mgd	$1,000,000
10 mgd and above	$0.20/gallon

1.7. Construction Cost Contingency

Construction cost estimates were based on planning level unit costs and include an allowance of 20% for construction contingencies.

2.0. Project Costs

Construction and replacement cost estimates presented in this Technical Memorandum were converted to total project costs by adding an allowance of 20% for engineering, legal and administrative fees. Project cost estimates are intended for use in budget development, wherever site-specific costs are not utilized. They represent typical experience and should be adjusted, where appropriate, to meet special needs.